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PROBABILISTIC APPROACH TO THE DESIGN AND TEST OF HARDENED FACIL--ETC(U)

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PROBABILISTIC APPROACH TO THE DESIGN AND TEST OF HARDENED FACILITIES.

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The general approach to probabilistic structural design is applied to the particular case of hardened ground facilities. The basic probabilistic concepts required for design are presented and illustrated by application of the methodology to the design of a composite steel/concrete liner for a deep-buried tunnel. An evaluation of the advantages and disadvantages of the probabilistic approach compared to the deterministic approach is made.

SUMMARY

The general approach to probabilistic structural design is applied to the particular case of hardened ground facilities. The basic probabilistic concepts required for design are presented and illustrated by application of the methodology to a composite steel/concrete liner for a deep-buried tunnel. An evaluation of the advantages and disadvantages of the probabilistic structural design approach compared to the deterministic design approach is made. For the example design, the allowable strength of the steel/concrete rock cavity lining is determined by Newmark's method which relates the free-field compressive stress to the various stresses and strains in the steel, concrete and rock. Multiple failure modes of the tunnel liner are accounted for when the applied loads result from a single source such as a nuclear burst.

Basic concepts of Bayesian statistical decision theory are presented along with two practical examples. One example illustrates a method for rationally selecting an acceptable failure probability for hardened facilities. The second example illustrates the application of statistical decision theory to the selection of the optimum test program for a deep-buried rock/liner structural system.

Recommendations are made for future research in the development of probabilistic design methodology in areas relating specifically to the design of deep-buried lined tunnels and which also have general applicability to other types of hardened facilities.

Table 1. Conversion factors for U.S. customary to metric (SI) units of measurement.

To convert from	To	Multiply by
mils	millimeters	0.0254
inches	centimeters	2.54
feet	meters	0.3048
miles	kilometers	1.6093
square inches	square centimeters	6.4516
square feet	square meters	0.0929
square miles	square meters	2,589,998.0
cubic inches	cubic centimeters	16.38706
cubic feet	cubic meters	0.0283
cubic yards	cubic meters	0.764555
gallons (U.S.)	liters	3.785
gallons (Imperial)	liters	4.542
ounces	grams	28.349
pounds	kilograms	0.454
tons (short)	kilograms	907.185
tons (long)	kilograms	1,016.047
pounds per foot	newtons per meter	14.59390
pounds per square inch	newtons per square centimeter	0.6894757
pounds per cubic inch	kilograms per cubic centimeter	27,679.90
pounds per square foot	newtons per square meter	47.88026
pounds per cubic foot	kilograms per cubic meter	16.0185
inches per second	centimeters per second	2.54
inch-pounds	meter-newtons	0.1129848
inch-kips	meter-kilonewtons	0.0001129848
Fahrenheit degrees	Celsius degrees or Kelvins[a]	5/9
kilotons	terajoules	4.183

a To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use C = (5/9) (F - 32). To obtain Kelvin (K) readings, use K = (5/9) (F - 32) + 273.15.

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SECTION I
INTRODUCTION

The use of probabilistic concepts in the structural design and testing of hardened facilities offers four significant benefits. First, it provides a framework in which inherently random quantities may be so considered, thus increasing the fidelity for analytical idealizations of loads and strengths. Second, it enables the various realistic estimates of uncertainties to be combined in a rational and consistent manner, thus producing meaningful estimates of the probability of structural failure. Third, it thereby permits a structural design with an explicit and reasonable balance between reliability and cost. Alternative deterministic procedures produce designs whose safety is neither balanced nor clearly specified (Reference 1). Several additional benefits of a probabilistic structural design approach relative to the conventional deterministic approach are discussed by Moses (Reference 2, page 84). For example, where the applied loading environment encompasses a broad spectrum, selecting the worst possible loading condition is "economically meaningless." A fourth factor is the necessity in design trade studies to compare optimized structures having different configuration and material. For this situation, a rational comparison is possible only if all the optimized structures have the same probability of failure. Some of these benefits will be demonstrated in the present report through a comparison of deterministic and probabilistic design approaches in a practical design example.

Basic probabilistic concepts required for the design example are presented in Section II. The hardened facility element used for the illustrative example is a composite steel/concrete liner for a deep-buried tunnel. The design data base available for this example is presented in Section III along with both deterministic and probabilistic design studies. Since the data and design quantities are given neither in consistent metric units nor in U.S.

customary units, conversion factors for U.S. customary to metric (SI) units of measurement are listed in Table 1. An approach to the use of probabilistic concepts in the structural testing of hardened facilities is included in Section IV. Conclusions of the present study and recommendations for future research activities are presented, respectively, in Sections V and VI. References used specifically in the present study and a selected bibliography which may be helpful in future studies are presented, respectively, in Sections VII and VIII. Details of the design equations and a computer implementation of the probabilistic design approach are included in the Appendix.

SECTION II
METHODOLOGY DEVELOPMENT

1. GENERAL

The four subjects in Section II concern several different methodologies associated with the design and test of hardened facilities. Paragraph 2 presents a brief general summary of probabilistic structural design criteria for both single components and assemblies consisting of several components. Detailed equations are presented for the survivability of an assembly having linearly dependent lognormal loads and independent lognormal strengths. A detailed discussion of the Taylor's series method for approximating the probability distribution of a nonlinear function of several independent random variables is also presented. The central limit theorem is discussed as a basis for using the lognormal probability law to represent the probability distributions of many analytical functions.

Paragraph 3 presents, as background material, a brief summary of the basic concepts of Bayesian statistical decision theory. An example of the application of decision theory involving choices between alternative systems and between alternative testing programs is also included. Paragraph 4 comprises a more detailed application of decision theory leading to the rational selection of failure probability comprising a balance between risk and cost. The use of utility theory to establish required "failure cost" data is discussed for a general hardened facility where significant monetary and nonmonetary consequences of failure exist. Paragraph 5 briefly illustrates a procedure for specifying design requirements consistent with a probabilistic structural design approach. This illustration is concerned specifically with a deep-buried hardened facility.

2. PROBABILISTIC STRUCTURAL DESIGN APPROACH

Fundamental concepts associated with probabilistic structural design are presented by Freudenthal in Reference 1. The probabilistic design approach proposed in Reference 1 is based on a recognition of the stochastic characteristics of both limit loads and strengths. The approach subsequently proposed by Ang and Amin (Reference 3) recognizes, in addition, the analytical uncertainties involved in evaluating these probabilistic loads and strengths. These uncertainties in analytically determining limit loads and strengths can be quantified by a constant coefficient of uncertainty (u) equal to or greater than unity. Thus the event $(S/L > u)$ constitutes a state of structural safety, where S and L represent the strength and limit load associated with a structural component. If S and L are both random variables, then the probability $P[S/L > u]$ is a proper measure of structural safety. The extended reliability structural design approach is then expressed by the following probabilistic equation for structural safety:

$$P\left[\frac{S}{L} > u\right] = 1 - P_F \tag{1}$$

where

- S is the random variable describing the component strength,
- L is the random variable describing the component limit load,
- u is the coefficient of uncertainty for the component, and
- P_F is the component probability of failure or acceptable risk.

When the limit-load and strength probability density functions are known, the structural design approach may be expressed in two equivalent forms:

$$P_S = \int_{-\infty}^{\infty}\int_{-\infty}^{\frac{y}{u}} f_L(x) \cdot f_S(y)\ dx\ dy \tag{2}$$

$$P_F = \int_{-\infty}^{\infty}\int_{-\infty}^{ux} f_S(y) \cdot f_L(x)\ dy\ dx \tag{3}$$

where

$P_S = 1 - P_F$ is the structural reliability,

$f_L(x)$ is the limit-load probability density function (PDF), and

$f_S(y)$ is the strength PDF.

The conventional factor of safety is defined as the ratio of the allowable strength (S_A) to the design limit load (L_D):

$$FS = \frac{S_A}{L_D} \tag{4}$$

where

S_A is the value of the random strength corresponding to a specified exceedance probability (P_A), and

L_D is the value of the random limit load corresponding to a specified non-exceedance probability (P_D).

The purpose of the factor of safety in the structural design procedure is to locate the strength PDF relative to the given limit-load PDF so that Equation (2) or (3) results in the required component reliability. This concept is illustrated in Figure 1. For most probability distributions, the integral of Equation (2) or (3) must be evaluated numerically and the required factor of safety determined by trial-and-error procedures. However, for certain specific distributions, closed-form evaluations leading to convenient design formulas are possible. A particularly convenient design factor-of-safety equation occurs when both limit loads and strengths are assumed to follow the lognormal probability law.

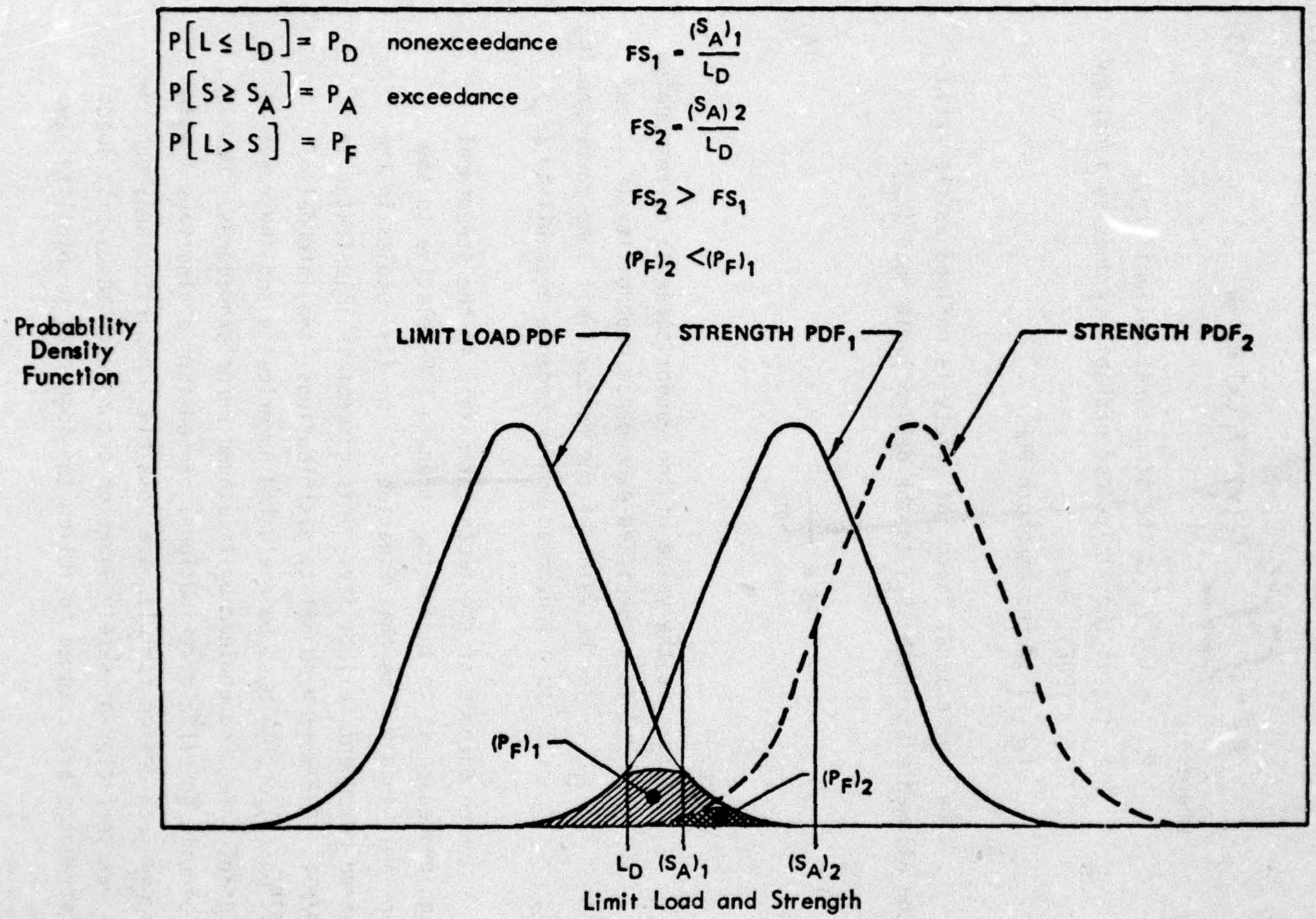


Figure 1. Graphical representation of factor of safety.

The component factor-of-safety expression for lognormal limit loads and strengths is derived in Reference 4 in the following form:

$$FS = \frac{S_A}{L_D} = u \cdot \exp - \left[F^{-1}(P_F)\sqrt{\ell n\left[(1+V_L^2)(1+V_S^2)\right]} + \right.$$

$$\left. F^{-1}(P_D)\sqrt{\ell n\,(1+V_L^2)} + F^{-1}(P_A)\sqrt{\ell n\,(1+V_S^2)} \right] \qquad (5)$$

where

- u is the coefficient of uncertainty,
- P_F is the probability of failure or acceptable risk,
- P_D is the non-exceedance probability for design limit load (L_D),
- P_A is the exceedance probability for allowable strength (S_A),
- V_L and V_S are limit-load and strength coefficients of variation, and
- $F^{-1}(P)$ is the inverse of the standardized normal cumulative distribution function given by

$$P = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{F^{-1}(P)} e^{-\frac{1}{2}t^2}\,dt \qquad (6)$$

The numerical behavior of the lognormal/lognormal factor of safety is shown graphically in Figure 2. For this plot, the defining probabilities for design limit load and allowable strength are both taken as 99 percent, and the coefficient of uncertainty is taken as unity. The factor of safety is seen to increase monotonically with decreasing probability of failure for given load and strength coefficients of variation.

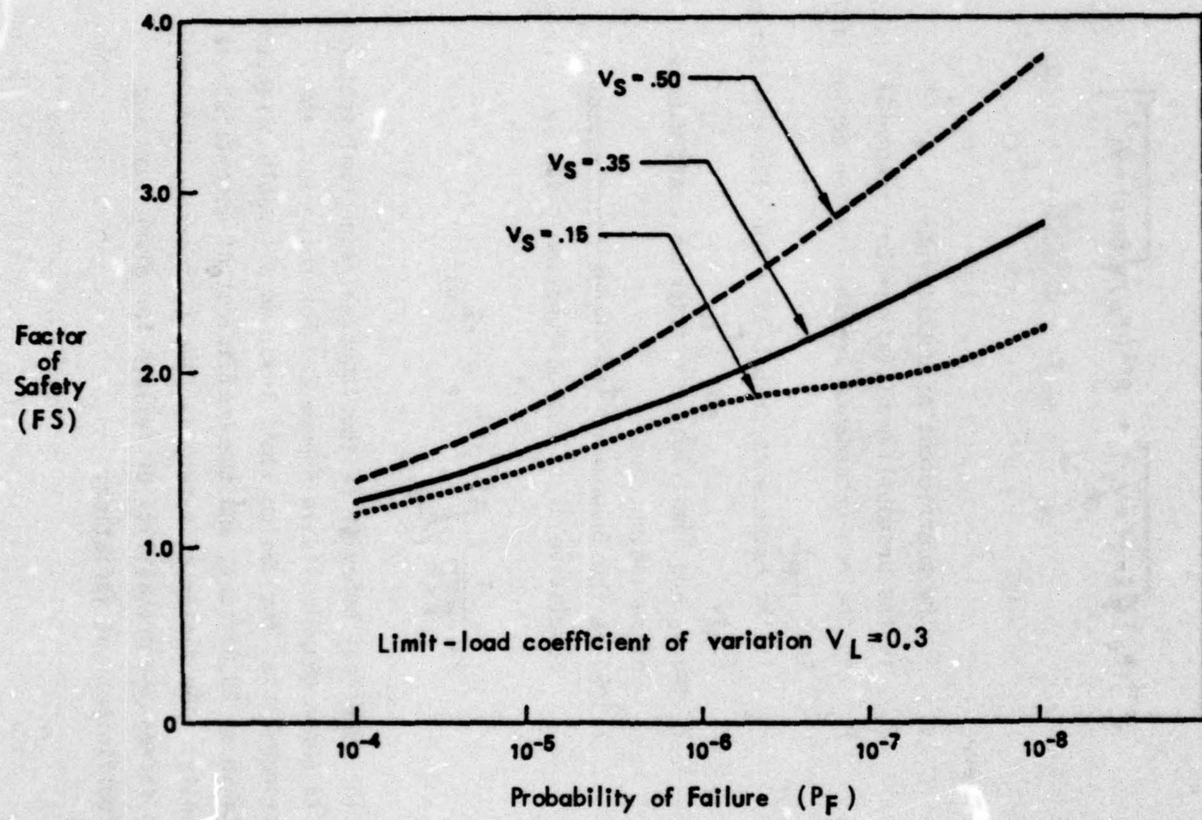


Figure 2. Factor of safety (FS) versus probability of failure (P_F).

From Equation (5), the component factor of safety corresponding to a specified reliability may be computed. The allowable strength is then determined, from Equation (4), as the product of the factor of safety times the design limit load. This approach is valid for an individual structure having a single failure mode where both the load and the strength follow the lognormal probability law.

A structure having multiple failure modes is termed an assembly. This basic design approach is also valid for individual components comprising an assembly when the component reliabilities are specified in terms of the assembly reliability and when the failure of a single component constitutes failure of the assembly (Reference 2, page 91). If all of the n component failure modes are statistically independent,

$$P_{Fa} = 1 - \prod_{i=1}^{n} (1 - P_{Fi}) \approx \sum_{i=1}^{n} P_{Fi} \qquad (7)$$

where

P_{Fa} is the probability of failure for the assembly, and

P_{Fi} is the probability of failure for the i^{th} component.

If either the component loads or the component strengths are correlated, the reliability of the structural design based on the approach of Equations (4) through (7) will be somewhat in error.

If the failure of a single component constitutes failure of the assembly, the assembly reliability is defined in terms of the simultaneous survival of each component. This "weakest-link" failure criterion appears adequate to describe the failure condition for a statically determinate structure and some evidence suggests its validity for statically indeterminate structures as well (Reference 5). Thus, for an assembly consisting of n components having

mutually correlated lognormal loads and strengths, the event ($S_1 > u_1 L_1$, $S_2 > u_2 L_2$, ..., $S_n > u_n L_n$) constitutes a state of structural safety. The reliability of the assembly is the probability of this event expressed in terms of the joint load and strength probability density functions in an analogous manner to Equation (2).

An important special type of assembly is one with all component loads linearly related and with all component strengths statistically independent. For this case, all the loads may be expressed in terms of a single variable and the general lognormal/lognormal reliability integral is greatly simplified. The resulting reliability integral, derived in Reference 4 and presented in Reference 2 (p. 90), is

$$P_S = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\infty} \exp\left(-\tfrac{1}{2} t^2\right) \cdot G_S(t)\, dt \tag{8}$$

where $$G_S(t) = \prod_{i=1}^{n} 1 - F(t_i) \tag{9}$$

$$t_i = \frac{\sigma_{Li} \cdot t + \mu_{uLi} - \mu_{Si}}{\sigma_{Si}} \tag{10}$$

The functions $F(t_i)$ represent cumulative probabilities of the standardized normal variate defined by Equation (6) and tabulated, for example, in Reference 6. The mean and standard deviation (μ and σ) of the transformed normal component limit loads and strengths are obtained from the actual lognormal mean and coefficient of variation (m and V) as follows:

$$\mu_i = \ell n \left(\frac{m_i}{\sqrt{1 + V_i^2}} \right) \tag{11}$$

$$\sigma_i = \sqrt{\ln\left(1+V_i^{2}\right)} \tag{12}$$

The approach of Equations (8) through (12) clearly requires iteration since trial values of the component load and strength parameters are necessary for the numerical evaluation of the reliability integral. Therefore, where the underlying assumptions are justified, this design approach is suited for optimization techniques (Reference 7).

The design approaches for independent components (Equations 4 through 7) and for assemblies with linearly dependent loads and independent strengths (Equations 8 through 12) require analytical relationships between load and strength since both load and strength must be expressed in directly comparable quantities. This often involves determining the probability distribution of a complicated nonlinear function of several random variables. A completely general method for obtaining the desired distribution is a numerical simulation of the function based on random sampling from the known or assumed probability distributions of the individual random variables. This method, known as the Monte Carlo method, was used as the basis for the probabilistic design studies of deep-buried hardened facilities described in Reference 8. For most practical applications, the Monte Carlo method is used to evaluate parameters of certain analytical distributions having theoretical justification. For example, the distribution of the largest value among n independent observations tends toward the extremal type I distribution (Reference 9, page 166) as n becomes large. A Monte Carlo study would be used merely to estimate the mean and standard deviation for the theoretical extremal distribution. Since the results of such a Monte Carlo analysis are treated statistically as measured data, the sampling variances of the parameters of interest decrease

as the number of replications increases. Standard statistical techniques provide measures of the accuracy of the estimates in terms of the number of replications. In addition, Reference 10 presents several practical techniques for reducing the sampling variance in Monte Carlo simulations.

While the Monte Carlo method is a powerful and general tool for obtaining the probability distribution of a complicated function, other less general methods may be preferable, where applicable, in preliminary design studies and in situations where precise and costly analyses are not warranted. One such approximate method, based on the use of Taylor's series, determines the distribution of a nonlinear function of several random variables by approximating the desired function as a linear function in the region of interest. A brief derivation of this method follows.

The mean and standard deviation of a linear function of several independent random variables are known from elementary probability theory (Reference 11, page 48). If X_1, X_2, ..., X_n are independent random variables having means m_1, m_2, ..., m_n and variances s_1^2, s_2^2, ..., s_n^2, respectively, and if a_1, a_2, ..., a_n are constants, then a linear random function may be defined as follows:

$$f(X_1, X_2, \ldots, X_n) = a_1 X_1 + a_2 X_2 + \ldots a_n X_n \tag{13}$$

The mean of f is

$$m_f = a_1 m_1 + a_2 m_2 + \ldots + a_n m_n \tag{14}$$

Thus the mean of a linear combination of random variables is equal to the linear combination of the means. This result is valid even if the X's are dependent.

The variance of f is

$$S_f^2 = a_1^2 S_1^2 + a_2^2 S_2^2 + \dots + a_n^2 S_n^2 \qquad (15)$$

Thus the variance of a linear combination of independent random variables is equal to the sum of the products of variances and squared constants.

A nonlinear function may be expanded in a Taylor's series about any given point as follows (Reference 11, page 62):

$$f(X_1, X_2, \dots, X_n) = f(m_1, m_2, \dots, m_n) + \sum_{i=1,n} (X_i - m_i)\, p_i$$

$$\text{where} \quad p_i = \left.\frac{\partial f}{\partial X_i}\right|_{m_1, m_2, \dots, m_n} \qquad (16)$$

If the higher order terms are negligible, the mean of f is, according to Equation (14), approximately equal to:

$$m_f \approx f(m_1, m_2, \dots, m_n) \qquad (17)$$

If in addition the X's are independent, the variance of f is, according to Equation (15), approximately equal to

$$S_f^2 \approx S_1^2 \left[\left.\frac{\partial f}{\partial X_1}\right|_{m_1, m_2, \dots, m_n} \right]^2 + \dots$$

$$+ S_n^2 \left[\left.\frac{\partial f}{\partial X_n}\right|_{m_1, m_2, \dots, m_n} \right]^2 \qquad (18)$$

The two fundamental assumptions in the use of the Taylor's series method are

(1) that the higher-order terms in the Taylor expansion are negligible compared with the first-order terms, and

(2) that the X's are independent.

The approximations involved in the Taylor's series method may be evaluated for a special case. Let the function of interest be a product of lognormal variates raised to arbitrary constant exponents as follows:

$$Y = \prod_{i=1}^{n} X_i^{C_i} \tag{19}$$

where

$$X_i \sim LN\,(m_i, V_i)$$

Modifying Equation (19) by the lognormal transformation

$$\ln Y = \sum_{i=1}^{n} C_i \ln X_i \tag{20}$$

where

$$\ln X \sim N\,(\mu_i, \sigma_i)$$

The relationships between the transformed normal parameters (μ and σ) and the lognormal parameters (m and V) are given in Equations (11) and (12). Since Equation (20) is now expressed in terms of normal random variables like Equation (13), the transformed mean (μ_Y) and standard deviation (σ_Y) may be obtained from Equations (14)

and (15). Performing the inverse logarithmic transformation implicit in Equations (11) and (12) results in the following exact expressions involving the mean (m_Y) and coefficient of variation (V_Y) of the basic function:

$$m_Y = \prod_{i=1}^{n} m_i^{C_i} (1+V_i^2)^{C_i/2\,(C_i-1)} \tag{21}$$

$$V_Y^2 = \prod_{i=1}^{n} (1+V_i^2)^{C_i^2} - 1 \tag{22}$$

Approximate relations for the mean and coefficient of variation of the product of independent lognormal random variables obtained from the Taylor's series method of Equations (17) and (18) are as follows:

$$m_Y \approx \prod_{i=1}^{n} m_i^{C_i} \tag{23}$$

$$V_Y^2 \approx \sum_{i=1}^{n} C_i^2 V_i^2 \tag{24}$$

The Taylor's series method provides a good approximation to the exact parameters when the C_i are close to unity and the V_i are small.

A useful property of the lognormal distribution is the multiplicative reproductive property: If X_1 and X_2 are independent lognormal variates, then the product $X_1 . X_2$ is also a lognormal variate. This result corresponds directly to the additive reproductive property of the normal distribution.

Perhaps the primary reason for the practical importance of the normal and lognormal distribution is the central limit theorem (Reference 12, p. 14). Just as sums of independent, arbitrarily distributed random variables tend to be normally distributed, so do products of independent, arbitrarily distributed random variables tend to be lognormally distributed. More specifically, let $\{X_j\}$ be a vector of independent, positive random variables of an unspecified distribution such that the following mean or expected values exist for every random variable.

$$E\left\{\ln X_j\right\} = \mu_j \tag{25}$$

$$E\left\{(\ln X_j - \mu_j)^2\right\} = \sigma_j^{\,2} \tag{26}$$

$$E\left\{\left|\ln X_j - \mu_j\right|^3\right\} = \omega_j^3 \tag{27}$$

Then if

$$\mu_n = \sum_{j=1}^{n} \mu_j \tag{28}$$

$$\sigma_n^{\,2} = \sum_{j=1}^{n} \sigma_j^{\,2} \tag{29}$$

$$\omega_n^{\,3} = \sum_{j=1}^{n} \omega_j^3 \tag{30}$$

the product $\prod_{j=1}^{n} X_j$ is asymptotatically lognormally distributed with transformed normal mean μ_n and transformed normal variance σ_n^2 provided that the ratio ω_n/σ_n approaches zero with increasing n. The actual lognormal mean and coefficient of variation are obtained from these transformed normal parameters by inverting Equations (11) and (12) as before. The convergence to the lognormal distribution is especially rapid, of course, if the individual variates have distributions similar to the lognormal distribution. Of particular interest in this regard, the normal distribution is essentially identical to the lognormal distribution when the coefficient of variation is small.

The probabilistic concepts discussed in this paragraph are the basis for the probabilistic design procedure described in paragraph 11. The structural reliability equations for components and assemblies (Equations 1 through 12) are used to determine the failure probability of a deep-buried facility. The parameters for the random allowable strengths of the facility components are determined from analytical functions of several random variables by means of the Taylor's series method (Equations 17 and 18). The allowable strengths are assumed to be lognormally distributed by virtue of the central limit theroem (Equations 25 through 30). The Taylor's series method and the exact expressions for products of lognormal random variables are also used in the discussion of probabilistic attack conditions (paragraph 6).

3. BAYESIAN STATISTICAL DECISION THEORY

Bayesian decision theory is a formal mathematical procedure for guiding a decision maker in the choice among various courses of action when the consequences of the choice are uncertain. The course of action recommended by the theory is one which is consistent with the decision maker's preference for the various consequences and with his judgements of the uncertainties involved.

According to Raiffa and Schlaifer (Reference 13), any decision problem in which experimentation is possible involves the following basic data:

1) a listing of the potential terminal acts $\{a\}$ which are available to the decision maker;
2) a listing of the possible states of nature $\{\theta\}$;
3) a listing of possible experiments $\{e\}$ which may be used to obtain additional information about the states of nature;
4) a listing of possible outcomes $\{z\}$ of these experiments;
5) a utility function which quantifies the decision maker's preferences for all (e,z,a,θ) combinations, and
6) a listing of the probabilities associated with the various experimental outcomes and states of nature.

A general analysis by Bayesian statistical decision theory permits the decision maker to consider each possible experimental outcome, to determine for each outcome the optimal terminal act, and then to use this information to choose an optimal experiment. Some brief introductions to the general topic of decision theory are contained in References 14 through 17.

Bayesian analysis presumes that the decision maker can assign a utility function to express his preferences among all possible consequences. As described by Luce and Raiffa (Reference 17), the utility function is used only to obtain a ranking of actions. Therefore any linear transformation of a utility function is equally valid since the ranking of actions remains unchanged. Values of the utility function may be expressed either in units of dollars gained or lost or in other, more general units when dollars do not reflect true utility.

Bayesian analysis also presumes that the decision maker can assign probabilities to each of the possible experimental outcomes and states of nature. These probabilities are based on whatever evidence, objective or subjective, is available. The concept of subjective probability reflects a subjective measure of the degree of belief the decision marker has in a judgement or prediction. If the decision maker has complete knowledge of an event so that he knows exactly what will happen, probability is a meaningless concept. The concept of probability is meaningful for incomplete knowledge in that it does represent a numerical summary upon which the decision maker is prepared to base his decision. (Reference 16)

In statistical decision theory, initial probabilities assigned to each possible state of nature are termed prior probabilities and designated $P'(\theta_i)$ for discrete states. If an experiment (e_k) is performed, the conditional "sampling" probability of discrete experimental outcome z_j given that θ_i is the true state of nature is designated $P(z_j|\theta_i)$. The unconditional sampling probability of z_j, $P(z_j|e_k)$ is obtained by averaging the conditional sampling probabilities using prior probabilities as follows:

$$P(z_j|e_k) = \sum_i P(z_j|\theta_i, e_k) \cdot P'(\theta_i) \tag{31}$$

The initial probabilities (assigned prior to the experiment) may be updated by means of Bayes' theorem if additional experimental evidence is obtained. The updated probabilities represent the conditional probability that θ_i is the true state of nature given that the outcome z_j occurred from experiment e_k. These probabilities, designated $P''(\theta_j|z_j,e_k)$, are termed posterior probabilities since they are assigned after the experiment. The posterior probabilities are obtained from Bayes' theorem as follows:

$$P''(\theta_j|z_j,e_k) = \frac{P(z_j|\theta_i,e_k) \cdot P'(\theta_i)}{P(z_j|e_k)} \qquad (32)$$

Several alternate procedures exist for assigning the probabilities associated with experimental outcomes and states of nature. For example, the unconditional sampling probability, $P(z_j|e_k)$, may be assigned directly along with the conditional state probability, $P(\theta_i|z_j,e_k)$. Nevertheless, the most common procedure is the direct assignment of $P'(\theta)$ and $P(z|\theta)$ for each experiment.

The rule used to determine the "optimum" decision which is usually adopted and used here is to choose the act which either maximizes the expected gain or minimizes the expected loss. Given a specific experiment (e_k) and a specific outcome (z_j), the optimal terminal act (a^*), which has the maximum expected gain, is determined using posterior probabilities defined by Equation (32).

$$\bar{u}(a^*|z_j,e_k) = \max_{e} \sum_i u(e_k,z_j,a,\theta_i) \cdot P''(\theta_i|z_j,e_k) \qquad (33)$$

This procedure is designated terminal analysis and is used to select the optimal terminal act after an experiment has been performed. The selection of the optimal experiment to be performed is guided by pre-posterior analysis. The expected gain of an experiment is determined by averaging the optimal terminal act for each

experimental outcome using unconditional sampling probabilities defined by Equation (31). The optimal experiment (e*) is, as before, the one having the maximum expected gain:

$$\bar{u}(a^*, e^*) = \max_{e} \sum_{j} \bar{u}(a^* | z_j, e) \cdot P(z_j | e_k) \qquad (34)$$

The general decision problem is described by Raiffa and Schlaifer (Reference 12) as a game between the decision maker and "chance".

Move 1: The decision maker selects an e in {e}.

Move 2: Chance selects a z in {z} according to the probability distribution $P(z|e)$.

Move 3: The decision maker selects an a in a .

Move 4: Chance selects a θ in $\{\theta\}$ according to the probability distribution $P''(\theta|z,e)$.

Payoff: The decision maker receives $u(e, z, a, \theta)$.

This sequence of moves may be represented graphically by a decision tree (Figure 3) when each choice has only a few options.

To illustrate these basic concepts of decision theory, consider the hypothetical choice between an existing system having known reliability and a new system to be designed to a somewhat higher reliability. Two different test programs, one ten times as costly as the other, are available, if desired, to provide additional evidence concerning the actual reliability of the new system. The total decision, then, involves not only the choice between systems but also the selection of the preferred test program.

The basic listings of possible acts, states of nature, experiments and outcomes are presented in Table 2. The known reliability of the existing system is the probability of state θ_1 occurring when act a_2 is selected. Let this known reliability be designated

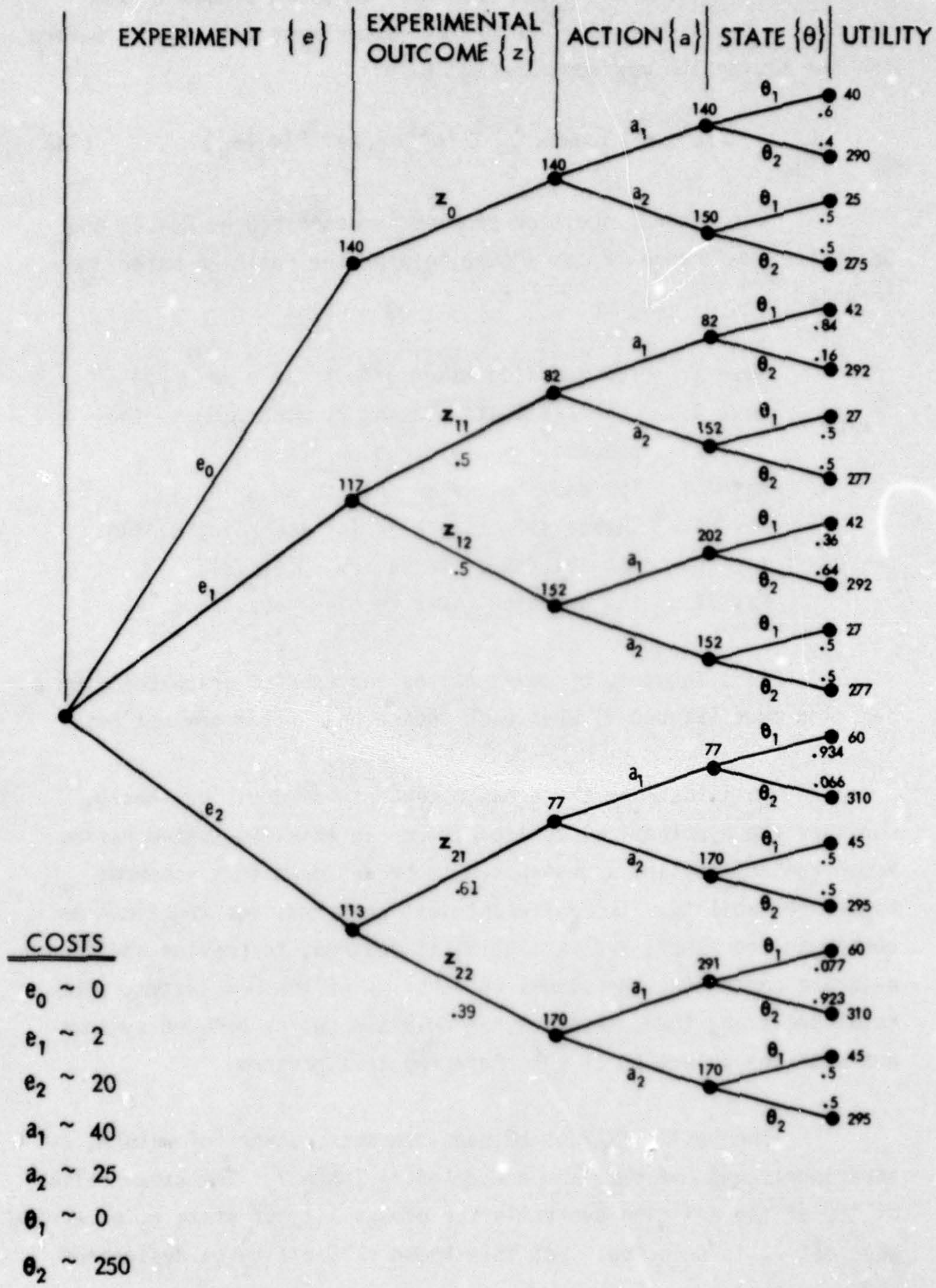


Figure 3. Decision tree: choice between designs and test program.

Table 2. Example description of options.

Space	Elements	Interpretation
terminal acts	a_1	accept new system design
	a_2	retain existing system design
states of nature	θ_1	system does not fail in service
	θ_2	system fails in service
experiments	e_o	no additional testing
	e_1	inexpensive test program
	e_2	expensive test program
outcomes	z_o	dummy outcome of e_o
	z_{11}	outcome of e_1 more favorable to θ_1
	z_{12}	outcome of e_1 more favorable to θ_2
	z_{21}	outcome of e_2 more favorable to θ_1
	z_{22}	outcome of e_2 more favorable to θ_2

$P'(\theta_1|a_2)$ and be 50 percent. Since one of the two states must occur, by definition, the failure probability of the existing system, $P'(\theta_2|a_2)$, is also 50 percent. Let the design reliability for the new system, which is the prior probability $P'(\theta_1|a_1)$, be 60 percent. The corresponding prior failure probability, $P'(\theta_2|a_1)$, is, of course, 40 percent. Thus the design reliability of the new system is somewhat greater than the known reliability of the existing system.

The three possible experiments consist of no testing (e_0), an inexpensive and relatively ineffective test program (e_1), and an expensive and relatively effective test program (e_2). The assumption is made, perhaps contrary to fact, that the test programs (e_1 and e_2) provide information which is directly related to the behavior of the new system in service. Neither of the test programs is infallible, but both provide useful information. The fallibility of the experiments is quantified in the conditional sampling probabilities. For e_1, let the probability that the test outcome favors θ_1 when θ_1 is the true state be $P(z_{11}|\theta_1,e_1)$ equal to 70 percent. The probability of the experiment e_1 indicating that the system will fail in service (θ_2) when θ_1 is the true state is, of course, 30 percent. Similarly, probabilities are assigned for both possible outcomes of e_1 when θ_2 is the true state. These probabilities for e_1 and the corresponding conditional sampling probabilities for e_2 are presented in Table 3 along with the prior probabilities for both possible terminal acts. The assigned prior and conditional sampling probabilities are used with Equations (31) and (32) to calculate the unconditional sampling probabilities and posterior probabilities also presented in Table 3. All the significant probabilities are listed on the decision tree of Figure 3 in their appropriate locations.

The costs for this hypothetical decision, are also presented in Figure 3. The individual costs of the experiments and the terminal acts are combined with the costs associated with the states of nature to result in total costs for all

Table 3. Example description of probabilities.

Prior Probabilities of State for Both System Designs

$P'(\theta_1 \mid a_1)$	= 0.6	$P'(\theta_1 \mid a_2)$	= 0.5
$P'(\theta_2 \mid a_1)$	= 0.4	$P'(\theta_2 \mid a_2)$	= 0.5
	1.0		1.0

Conditional Sampling Probabilities for New System Design

$P(z_{11} \mid \theta_1, e_1)$	= 0.7	$P(z_{21} \mid \theta_1, e_2)$	= 0.95
$P(z_{12} \mid \theta_1, e_1)$	= 0.3	$P(z_{22} \mid \theta_1, e_2)$	= 0.05
	1.0		1.00
$P(z_{11} \mid \theta_2, e_1)$	= 0.2	$P(z_{21} \mid \theta_2, e_2)$	= 0.1
$P(z_{12} \mid \theta_2, e_1)$	= 0.8	$P(z_{22} \mid \theta_2, e_2)$	= 0.9
	1.0		1.0

Unconditional Sampling Probabilities for New System Design
(Equation 31)

$P(z_{11} \mid e_1)$	= 0.5	$P(z_{21} \mid e_2)$	= 0.61
$P(z_{12} \mid e_1)$	= 0.5	$P(z_{22} \mid e_2)$	= 0.39
	1.0		1.00

Table 3. Example description of probabilties. (continued)

Posterior Probabilities of State for New System Design
(Equation 32)

$p''(\theta_1 \mid z_{11}, e_1)$	=	0.84	$p''(\theta_1 \mid z_{21}, e_2)$	=	0.934
$p''(\theta_2 \mid z_{11}, e_1)$	=	$\underline{0.16}$	$p''(\theta_2 \mid z_{21}, e_2)$	=	$\underline{0.066}$
		1.00			1.000
$p''(\theta_1 \mid z_{12}, e_1)$	=	0.36	$p''(\theta_1 \mid z_{22}, e_2)$	=	0.077
$p''(\theta_2 \mid z_{12}, e_1)$	=	$\underline{0.64}$	$p''(\theta_2 \mid z_{22}, e_2)$	=	$\underline{0.923}$
		1.00			1.000

(e, z, a, θ) combinations as shown. The expected utilities for each action are calculated for all experiments and experimental outcomes, as shown in Figure 3. The terminal analysis of Equation (33) then permits a selection of the optimal terminal act for each experiment and outcome. For example, if no testing is performed (e_0), the optimal act is to accept the new system design (a_1) since its expected cost is less than that for a_2:

$$\bar{u}(e_0, z_0, a_1) = 140 < \bar{u}(e_0, z_0, a_2) = 150$$

Notice that for e_1 and e_2, the optimal act is clearly determined by the test result.

The expected costs corresponding to the five outcomes are now used with the unconditional sampling probabilities to select the optimal experiment. This preposterior analysis is accomplished with Equation (34). The results favor the expensive test program (e_2) as shown in Figure 3. For this example, the choice of e_2 is preferable when the failure cost exceeds 200. When the failure cost is less than 200, the preferred choice is the inexpensive test program (e_1).

4. SELECTION OF FAILURE PROBABILITY

The problem of determining the least-cost design with maximum possible reliability may be approached in at least three different ways:

(1) minimize the total expected cost;
(2) minimize the failure probability subject to an allowable total cost;
(3) minimize total cost subject to an acceptable failure probability.

Since the determination of acceptable failure probability for hardened facilities requires a detailed understanding of national goals and priorities not generally available to a facility designer, a cognizant government agency should be responsible for this determination. The recommended approach for this agency to use in determining the acceptable failure probability is the minimum total expected cost hypothesis of approach (1). Given the resulting acceptable failure probability, the facility designer may then minimize total cost as in approach (3). Moses and Kinser (Reference 18) demonstrate appropriate optimization techniques in the analogous problem of minimizing total structural weight subject to an acceptable failure probability.

The minimum expected cost hypothesis of statistical decision theory has been used by Turkstra (Reference 19), Lind and Davenport (Reference 20), Freudenthal (Reference 1), Shinozuka et al (Reference 21), and others to select an acceptable failure probability for a structural scheme. A scheme is defined herein as a set of structures which are essentially identical except in initial cost and failure probability. Therefore, except for initial cost, all structures within a scheme would satisfy the intended purpose equally well if successful and would lead to identical results if unsuccessful.

All structures within a given scheme also must have in common a single type of failure, such as catastrophic failure, even though multiple failure modes are possible. The failure condition may be defined for an assembly by the "weakest-link" criterion; that is, failure of the assembly may be assumed to occur when any of its components fails. With the restriction to a single type of failure, the only possible states of nature which describe the structural behavior are success or failure. The total expected cost of the i^{th} alternative structure within a given scheme is then the sum of the expected success cost and the expected failure cost:

$$C_{Ti} = C_{Ii} \cdot (1 - P_{Fi}) + (C_{Ii} + C_F) \cdot P_{Fi}$$

$$= C_{Ii} + C_F \cdot P_{Fi} \qquad (35)$$

where

C_{Ti} is the total expected cost of the i^{th} structure.

C_{Ii} is the initial (life-cycle) cost of the i^{th} structure.

C_F is the net failure cost of the scheme.

P_{Fi} is the acceptable failure probability for the i^{th} structure.

The failure probability and the initial cost are inversely proportional and are related through the probabilistic structural design approach. Thus an increase in initial cost is necessary for a decrease in failure probability. Failure probability and initial cost are related analytically since both are dependent on the design parameters.

For any typical hardened facility, the success cost and the net failure cost may not be known with certainty. For this situation, Raiffa and Schlaifer (Reference 13, p. 17) recommend that

the state space be expanded to include uncertainty in cost as well as in structural behavior. Since the probability measure on the cost estimates is independent of the structural behavior, the various possible discrete costs may be replaced by the mean values of the initial and net failure costs. This significant simplification is shown schematically in the decision tree of Figure 4. The approximation in defining the mean failure cost with the initial cost held constant is considered acceptable since C_F is significantly greater than C_I for practical structures. Thus, Equation (35) may be considered valid even when precise cost estimates are unavailable so long as the mean values for the initial and net failure costs are used as certainty equivalents for the random cost estimates.

The failure probability corresponding to minimum total expected cost has been determined by Lind and Davenport (Reference 20) for a special case having wide applicability. For this case, the natural logarithm of the failure probability is assumed to be very nearly a linear function of initial cost in the interesting range of high reliability. This relationship may be written

$$P_F \approx b \cdot \exp\left(\frac{-C_I}{c}\right) \qquad (36)$$

where

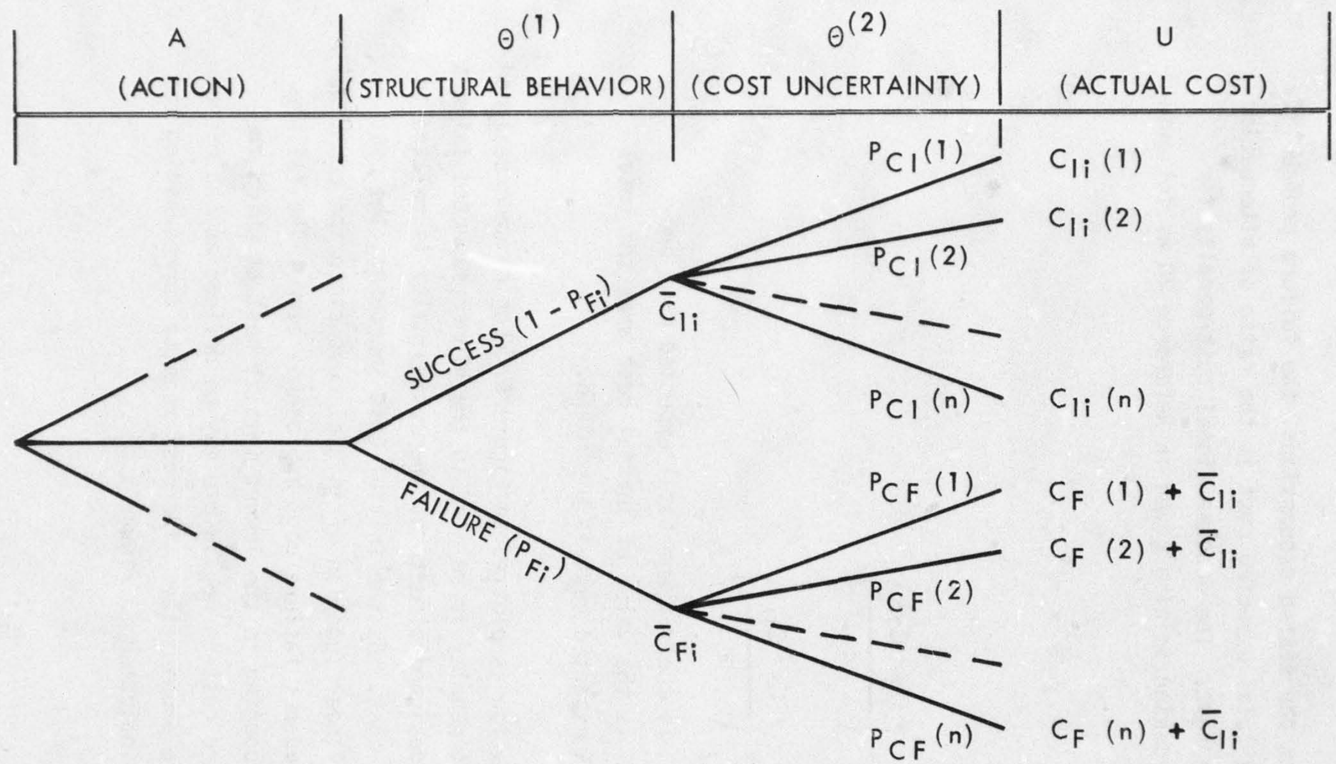
C_I is the (mean) initial cost

c is the attenuation cost defined as the slope of the initial cost vs. logarithm of P_F, and

b is a proportionality constant.

The attenuation cost or cost slope is seen to be the cost of reducing the failure probability by a factor of e($\approx$ 2.718...). Substituting Equation (36) into Equation (35) and minimizing the total expected cost results in the following relation for optimum failure probability:

$$P_{F_0} = \frac{c}{C_F} \qquad (37)$$



$$\overline{C}_{Ii} = \sum_{i=1}^{n} P_{CI}(i) \cdot C_{Ii}(i)$$

$$\overline{C}_{Fi} = \sum_{i=1}^{n} P_{CF}(i)\left[C_F(i) + \overline{C}_{Ii}\right] = \overline{C}_F + \overline{C}_{Ii}$$

Figure 4. Decision tree using certainty equivalents for unknown costs.

Thus, with the stated assumptions, the failure probability which minimizes the total expected cost is the ratio of attenuation cost to net failure cost. The nondimensional cost penalty for underdesign or overdesign is also given in Reference 20 as follows:

$$y = x - 1 + e^{-x} \tag{38}$$

where

$$y = \left(\frac{C_T - C_{T0}}{c}\right)$$

$$x = \left(\frac{C_I - C_{I0}}{c}\right)$$

C_{To} is the minimum total expected cost and

C_{Io} is the value of initial cost when the total expected cost is minimized.

This relation is plotted in Figure 5. For an underdesigned structure, the cost penalty is related to the higher expected failure cost. For an overdesigned structure, the cost penalty is related to the higher initial cost. As the initial cost increases, the cost penalty becomes a linear function of initial cost since the contribution of the expected net failure cost approaches zero. The failure probability corresponding to the lowest initial cost at which the expected net failure cost is negligible may be defined as the effectively zero failure probability. The cost penalty corresponding to this condition is indicated in Figure 5.

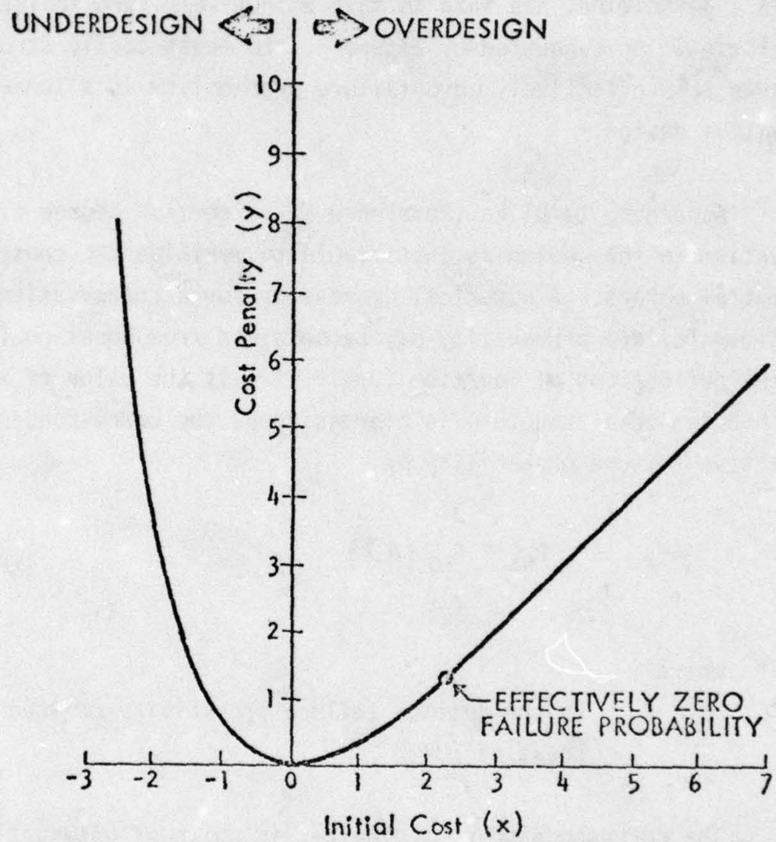


Figure 5 Cost penalty for underdesign or overdesign

A somewhat different definition of effectively zero failure probability is presented by Turkstra (Reference 18) for the case that initial costs or net failure costs have limited accuracy. When the individual cost terms in Equation (35) are rounded to an appropriate level of accuracy, the expected net failure cost for some alternatives can be rounded to zero. These alternatives, according to Turkstra's definition, are said to have effectively zero failure probabilities. As suggested by Figure 5, the least costly structure with Turkstra's effectively zero failure probability is a somewhat conservative design.

According to Blake (Reference 22) a certain degree of conservatism in the design is justifiable to minimize the consequences of potential errors. A numerical expression for a conservative, non-optimum failure probability may be obtained from Equation (36) using the definitions of Equation (38). If λ is the value of x in Figure 5 by which a structure is overdesigned, the corresponding conservative failure probability is

$$P_{F\lambda} = P_{FO}\left(e^{-\lambda}\right) \tag{39}$$

where

P_{Fo} is the optimum failure probability defined by Equation (37).

The corresponding cost penalty, in units of attenuation cost (c), is obtained by substituting λ for x in Equation (38). A reasonable approximation to the effectively zero failure probability is obtained by setting $\lambda = 2.3$. For this value, the conservative failure probability is 10 percent of the optimum failure probability and the cost penalty is 1.4 attenuation cost units.

For a numerical example of this approach, consider the optimum failure probability for a deep-buried facility in granodiorite to protect part of the field headquarters of a military unit. Assume that loss of the facility would result in reduced effectiveness of the unit and replacement costs for personnel and equipment. Let this net failure cost be expressed as 5.0×10^6 dollars. Assume the initial cost of the facility is the following function which is dependent only on burial depth:

$$C_I = 4.2 \times 10^6 - 695\,D + 2D^2 + 7240\,D^{0.08}$$

This includes 3.6×10^6 dollars for construction of the facility after the composite steel/concrete liner is complete. By means of the probabilistic structural design approach described in paragraph 11, a functional relationship can be established between initial cost and failure probability. Assume that a probabilistic design study results in the following functional relationship:

$$P_F = 79.652 \times 10^6 \exp\left(\frac{-C_I}{0.216 \times 10^6}\right)$$

The optimum failure probability determined from Equations (36) and (37) is simply

$$P_{FO} = \left(\frac{c}{C_F}\right) = \frac{0.216 \times 10^6}{5.0 \times 10^6} = 0.0432$$

Thus, for this example, the optimum failure probability is approximately 4.3 percent. The initial cost for this design is 4.6×10^6 dollars which corresponds to a burial depth of approximately 650 meters. The effectively zero failure probability is then approximately 0.43 percent. The burial depth of approximately 875 meters corresponds to an increase in initial cost of approximately 0.5×10^6 dollars.

The primary difficulties in applying the minimum expected cost hypothesis are concerned with the determination of initial costs and net failure costs. Presumably, suitable data can be generated for estimating initial (life-cycle) costs of proposed hardened facilities. According to Agbabian Associates (Reference 23, page I-11), these life-cycle costs should include the present worth of the development, capitalization, operations, and maintenance costs. To the extent that these costs are constant for all the alternative structures within a scheme, the costs may be neglected. As shown in Equation (36), this simplification is justified because the attenuation cost is unaffected by a change in the proportionality constant.

Incidentally, while the present expected cost model was not originally intended as a system cost-effectiveness model for comparing alternative schemes, it may be so used if such factors as technical effectiveness and implementation period can be converted to equivalent initial costs. One may observe that "implementation period" is already converted to "equivalent dollars" through the incentive and penalty clauses of construction contracts. This fact suggests that perhaps applicability of the expected cost model may rationally be extended to system analysis.

Determining the net failure cost associated with the loss of a hardened facility under attack is a difficult and controversial task. This is especially true for situations in which nonmonetary consequences of failure are much more significant than such monetary considerations as replacement costs. The use of questionnaires to determine interpretations of national goals and priorities by appropriate government decision makers appears essential to this task. The successful use of such questionnaires to gather expert opinion in the field of structural analysis and testing is described in Reference 24 (Appendix B).

In Reference 25 (page 91), Turkstra suggests defining the net failure cost directly in dollars by using Equation (35) along with the results to a question of the form: "If a failure probability of 10^{-4} leads to an initial cost of 10^{9} dollars, what increase in initial costs would you accept to reduce the failure probability to 10^{-5}?" This approach appears difficult to implement in practice because of problems with relating such small probabilities to practical experience.

An alternate approach consists of expressing both initial cost and net failure cost in terms of reduction in national capability.* For initial costs, the alternate approach involves defining a linear relationship between the utility of dollars and the utility of national capability. The resulting linear utility function could then be used to convert estimated life-cycle costs in dollars into units of equivalent national capability. Evaluating utilities by this indirect method is recommended by Raiffa and Schlaifer (Reference 13, page 80) for situations having mixed monetary and nonmonetary consequences. The required utility function could be obtained from a "least-squares" fit to the results of several questions of the form: "By what percentage could the national capability be increased by the present expenditure of X dollars?" The term "national capability" would, of course, require a precise and generally accepted definition.

For net failure costs, the alternate approach involves averaging the results of a question which uses the concept of a lottery.

"Consider the following three events:

A $\sim$ total national capability with the proposed facility,

B $\sim$ reduced national capability if the facility is lost during attack,

C $\sim$ no effective national capability.

---

* This suggestion is due to Carl Bagge of Agbabian Associates.

Now, consider the following two options:

(1) event B for certain, or

(2) a lottery involving event A with probability (1-p) and event C with probability p.

Note that if p is close to zero, the lottery is your preferred option since the most desirable event (A) is likely to occur. And if p is close to unity, the certain event (B) is preferable to the likely occurrence of the most undesirable event (C). As p changes from 0 to 1, a preference for the lottery option must change into a preference for the certain option. What is the value of p at which you are indifferent to the two options?"

At the indifference point, the expected utilities between the two options are equal:

$$U(A)\cdot(1-p) + U(C)\cdot p = U(B)\cdot 1 \tag{40}$$

If event A has utility 1 and event C has utility 0, then by Equation (40) the utility of event B is equal to 1-p. In units of national capability, the net failure cost is therefore numerically equal to p. This general approach to assigning utilities in terms of preferences between alternatives is included in the detailed discussion of utility theory in Reference 17 (Section 2).

The use of questionnaires appears feasible to evaluate both the utility of dollars in terms of national capability and the utility of reduced national capability in event of failure of the proposed facility. Using average values of the experts' answers to the questions also appears consistent with the use of certainty equivalents for unknown costs as outlined in Figure 4. Thus the selection of acceptable failure probability using the minimum expected cost hypothesis for hardened facilities does appear feasible and worthy of further study.

5. PROBABILISTIC DESIGN REQUIREMENTS

The specification of design requirements in a probabilistic manner is consistent with the application of probabilistic structural design concepts to deep-buried hardened facilities. The explicit evaluation of failure probability as a function of cost can lead to the optimum use of resources. Studies for determining the probability of failure may be performed using techniques such as those described in paragraph 4. Since attack conditions are not known with certainty, design requirements should be realistically specified as required failure probability and as probabilistically described attack conditions. The following indicates a convenient procedure for including probabilistic attack conditions in the structural design of deep-buried hardened facilities.

For a deep-buried hardened facility, the peak transient free-field compressive stress at depth due to a surface nuclear burst is influenced greatly by the transmissibility of the stress waves through the earth. Since the transmissibility characteristics of in situ earth materials cannot be known with certainty, a statistical description of these characteristics is appropriate. Such descriptions are presented by Perret and Bass in Reference 26 for various rock types. These descriptions are of the form

$$P_o' = A_o \left(\frac{R}{W^{1/3}} \right)^{\gamma} \tag{41}$$

where

P_o' = free-field stress at depth in pounds per square inch (psi)

W = weapon yield (underground) in kilotons (KT)

R = range in meters.

The coefficient A_0 is presented as a lognormal random variable describing rock transmissibilities. The exponents are assumed constant. A somewhat more general formulation of Equation (41) is

$$P_0' = A_0 (K_W W_S)^{\alpha} (D^2 + LR^2)^{\beta} \tag{42}$$

where

P_0' = free-field stress (psi) at depth
W_s = weapon yield (KT) at surface
K_w = weapon effectiveness factor or coupling factor
D = depth in meters, and
LR = lateral range in meters.

Again, A_0 is lognormal and the exponents are assumed constant.

Since the rock transmissibilities are represented probabilistically in Equation (42), specifying the attack conditions (K_W, W_S, and LR) similarly seems consistent and appropriate. Specifying a deterministic attack condition for design seems unnecessarily restrictive. The uncertainties associated with weapon yield, effectiveness of a near-surface burst in generating stress waves in the rock, and lateral range at detonation (i.e., miss distance of the attacking weapon) are significant and can be realistically included in the design process.

The uncertainties associated with the attack conditions may be combined with the random rock transmissibilities using either the first-order Taylor's series approximation or, somewhat more accurately, the product of lognormal random variables. Both methods are described in paragraph 2. From the latter approach, the mean value of the lognormal free-field compressive stress of Equation (42) is

$$\overline{P_0'} = \overline{A_0} (\overline{K}_W \overline{W}_S)^{\alpha} (\overline{D}^2 + \overline{L}R^2)^{\beta} (1 + V_{KW}^2 + V_{WS}^2)^{\alpha/2 (\alpha-1)} (1 + V_X^2)^{\beta/2 (\beta-1)} \tag{43}$$

The corresponding coefficient of variation is obtained from

$$V_{P_O'}^2 = (1+V_A^2)(1+V_{KW}^2+V_{WS}^2)^{\alpha^2}(1+V_X^2)^{\beta^2}-1 \tag{44}$$

The coefficient of variation (V_X) of the squared slant range is adequately represented by the first-order Taylor's series approximation:

$$V_X = 2\left[\left(\frac{\overline{D}^2}{\overline{D}^2+\overline{LR}^2}\right)^2 V_D^2 + \left(\frac{\overline{LR}^2}{\overline{D}^2+\overline{LR}^2}\right)^2 V_L^2\right]^{1/2} \tag{45}$$

The facility burial depth (D) has a relatively small uncertainty which is determined primarily by the terrain elevations. The lateral range (LR) is often represented by the Rayleigh distribution (Reference 27, page 320). Since the Rayleigh distribution is adequately represented by the lognormal probability law, the squared slant range is approximately lognormally distributed. For the Rayleigh distribution, the mean lateral range is related to the median range as follows:

$$\overline{LR} = 1.0645\ L_{0.5} \tag{46}$$

The coefficient of variation of a Rayleigh distribution in constant (0.5227), thus the coefficient of variation for the lateral range is:

$$V_{LR} = 0.5227 \tag{47}$$

Thus the probabilistic attack conditions may be conveniently and realistically included in the design process by the use of Equations (43) through (47).

SECTION III
STRUCTURAL DESIGN APPLICATION

6. GENERAL

The probabilistic structural design concepts described in paragraph 2 are illustrated in a practical design example. The example hardened facility element, a composite steel/concrete liner for a deep-buried tunnel, is described in paragraph 7. The analytical approach for the deterministic design of a rock/liner tunnel system is presented in paragraph 8. The data base required for either deterministic or probabilistic design studies is presented in paragraph 9 while the studies themselves are presented in paragraphs 10 and 11. A comparison between the deterministic and probabilistic structural design approaches, for the deep-buried tunnel example, is included in paragraph 12.

7. DESCRIPTION OF EXAMPLE HARDENED FACILITY ELEMENT

The example chosen to illustrate the probabilistic design approach is a composite steel and concrete liner for a deep underground facility in rock. The liner is for a hypothetical deep-buried facility, and the example is not intended to represent any particular past or future hardened defense facility. However, the example facility is assumed to be of sufficient strategic importance to justify a conservative design. This design example is concerned with the characteristics of the steel plate and concrete rock reinforcement, with the required depth of burial, and with the corresponding construction costs. The example therefore has all of the significant parameters affecting the design of an actual deep-buried hardened facility.

Figure 6 illustrates a liner cross-section for a drift in the hypothetical defense facility. The equipment and personnel within the rock cavity liner are assumed to be shock isolated. The rattlespace envelope around the isolation system and required equipment is assumed to accommodate noncatastrophic liner deformations. Therefore, catastrophic failure of the liner is the critical system design condition. The tunnel diameter of 14 feet is considered typical of part of a defense system. The length of the drifts becomes very important to the liner design when costs are considered. If a drift is short, the optimum design has the strongest possible liner in order to reduce the burial depth. However, if a drift is long, liner costs are relatively more significant and a greater burial depth provides a more economical option.

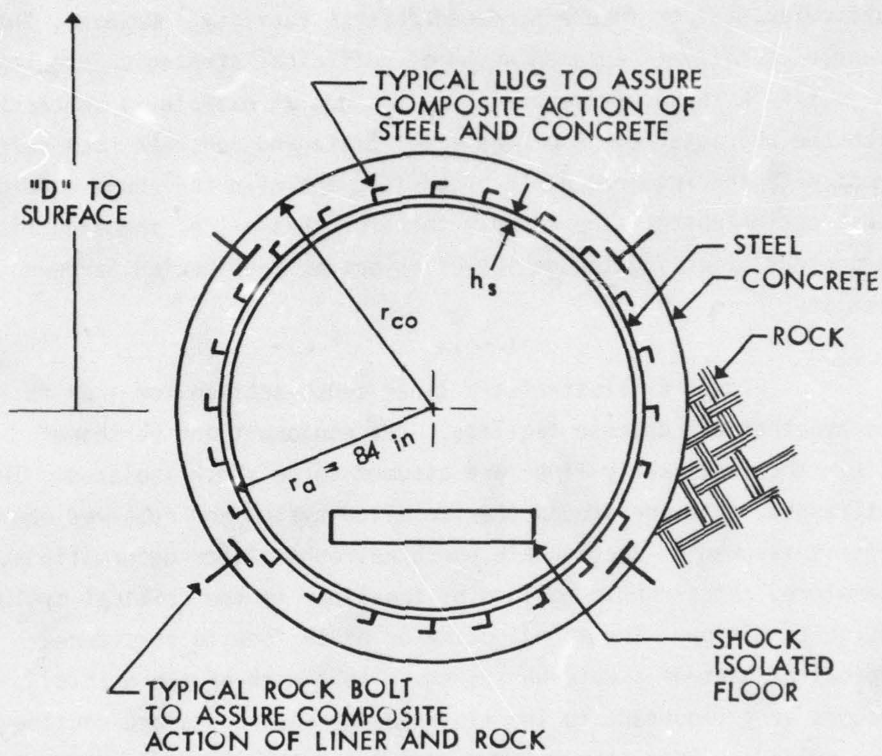


Figure 6. Deep-buried hardened facility example.

For this example design, the threat is assumed to be a single direct hit from a 25 megaton surface burst. The weapon effectiveness factor (kw) is assumed to be 0.04. This factor describes the effectiveness of a surface burst in generating stress waves in the rock. Probabilistic descriptions of miss distance and weapon yield are not necessary to illustrate the design approach. Such probabilistic attack conditions could easily be incorporated as discussed in paragraph 6. For the probabalistic design studies described in paragraph 11, single acceptable failure probability was not specified explicitly. A range of failure probabilities was assumed in the probabilistic parametric design studies.

8. DETERMINISTIC DESIGN APPROACH

For both deterministic and probabilistic structural design approaches, analytical relationships are required for relating applied loads (P'_o) with allowable strengths (P_o). For deep-buried hardened facilities, the applied load is generally described by the Perret-Bass data (Reference 26) in terms of free-field compressive stress at depth (P'_o). The allowable strength of steel/concrete rock cavity linings is generally determined by Newmark's method (Reference 28, paragraph 8.11.2). This method relates the free-field compressive stress (P_o) to the various stresses and strains in the steel, concrete, and rock. Figure 7 illustrates the significant stresses in the rock/liner structural system. σ_{ra} is the radial confining pressure applied to the concrete liner by the steel liner. σ_{rc} is the radial confining pressure applied to the rock by the concrete liner.

The various simplifying assumptions inherent in Newmark's method include the following:

(a) The analysis of the concrete/steel liner and surrounding rock may be treated as a plain strain problem. This obviously excludes tunnel ends and intersections.

(b) The dynamic response of the tunnel liner to the applied transient free-field compressive stress may be treated as quasi-static. This assumes that the free-field pressure waveform has slow rise and decay times relative to the natural period of the structure.

(c) The applied loading and the resulting stresses and strains may be treated as having radial symmetry. This requires that K = 1 (in Figure 7) for the lateral stress and that no bending occurs in the liner.

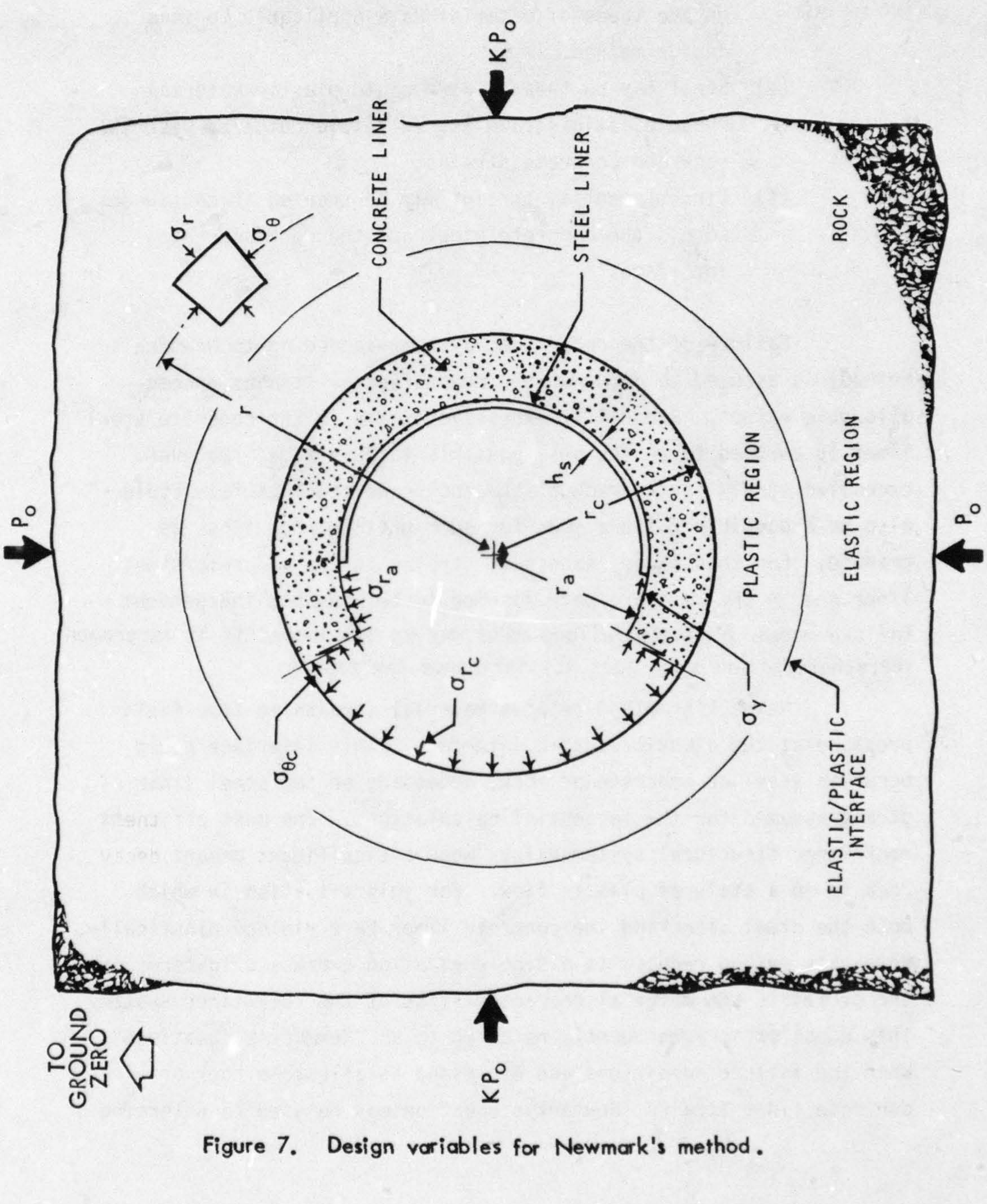


Figure 7. Design variables for Newmark's method.

(d) Rock and concrete may be treated as linearly-elastic/perfectly-plastic materials which follow the Mohr-Coulomb strength criterion. This imposes requirements on the types of material data applicable to this design method.

(e) Steel may be treated as a rigid-plastic material whose elastic strain is negligible compared with the rock and concrete strains.

(f) Circumferential strains may be treated as continuous across the concrete/steel and the rock/concrete interfaces.

Failure of the rock/liner system, according to Newmark's method, is assumed to occur when circumferential strains exceed allowable values. Generally, excessive strain in the concrete/steel liner is assumed to be the only possible failure mode. However, excessive strain in the rock at the rock/concrete interface could also be a possible failure mode for such brittle rock types as granite. For this study, excessive strains in the concrete/steel liner and in the rock are both assumed to be possible independent failure modes. A single failure mode may be considered if it is proven the other failure mode does not influence the result.

Newmark's method relates material strains to free-field pressure at the elastic/plastic interface. This interface could occur in steel or concrete or rock, depending on the steel liner strain assumed for the sequential calculations. The most efficient rock/liner structural system exists when a significant amount of rock is in a state of plastic flow. For this situation in which both the steel liner and the concrete liner have yielded plastically, Newmark's method reduces to a single equation expressed in terms of the geometric and material characteristics of the rock/liner system. This equation is subsequently referred to as "Newmark's equation." When the failure conditions are expressed as allowable rock or concrete liner strains, Newmark's equation may be used to determine

the corresponding allowable free-field compressive stresses in the rock. The equations for allowable free-field stress (P_o) in terms of the two possible failure strains are presented in Appendix A.1.

Although Newmark's method is used exclusively in this study, other methods are available for relating free-field stress to rock/liner system failure. One such method, documented in Reference 29, is similar to the Newmark method except that the assumption of a hydrostatic stress distribution is not required. Solution of the general equations with radial symmetry (K = 1) provides the same stresses and strains as Newmark's method. Reference 29 presents these general equations along with results of tests using small-scale lined tunnels. Another alternative method is the finite-element method which is probably the most versatile approach for relating free-field stress to rock/liner system failure. Such numerical solutions do not require the limiting assumptions inherent in the closed-form solutions. Material mechanical properties can be modeled as accurately as test data permit. Sliding boundaries may be used to represent rock jointing. The major drawback of the finite-element solutions compared with the closed-form solutions is the significant increase in computer time and the lack of visibility of the effects of particular design parameters. In general, the closed-form solutions are recommended for preliminary design studies and the finite-element simulations are recommended for final design verification.

For convenience in evaluating the construction cost of deep-buried tunnels having composite steel/concrete liners, a general cost function has been determined as follows:

$$CI = D\,(MM + BB \cdot D + C) + R\Big[E(r_a \cdot RCA)^2 \cdot D^F + C + G \cdot r_a \cdot h_s + H \cdot r_a^2\,(RCA^2 - 1)\Big] \qquad (48)$$

where

CI ~ construction cost (dollars)

D ~ tunnel depth (meters)

R ~ tunnel length (meters)

r_a ~ outer radius of steel liner (inches)

RCA ~ ratio of concrete outer radius to steel outer radius

h_s ~ thickness of steel liner (inches)

MM ~ cost parameter for access shaft excavation

BB ~ cost parameter for access shaft excavation

C ~ cost parameter for venting

E ~ cost parameter for main tunnel excavation

F ~ cost parameter for main tunnel excavation

G ~ cost parameter for steel liner

H ~ cost parameter for concrete liner

The cost parameters are established from existing cost data and contractors' estimates. The estimated construction cost for any given set of design parameters (D, R, r_a, and RCA) can then be determined from Equation (48). Other design parameters, such as concrete strength, could be included in the cost equation for more detailed estimates.

9. DATA BASE

9.1 Applied Load Data

The site location for the hypothetical facility is assumed to be either in soft rock (wet tuff) or in hard rock (granodiorite). The applied load data for both types of rock are obtained from the Perret-Bass data (Reference 26). These data consist of measurements from underground tests grouped into categories of rock type. The applicable results are presented in plots of peak free-field particle velocity versus normalized range from the weapon point. Normalized range is defined as the actual range in meters divided by the actual weapon yield in kilotons. Peak free-field stress is related to velocity by:

$$P_o' = \rho c v \tag{49}$$

where

ρ = rock density
c = wave speed
v = peak free-field particle velocity

The measured data from 14 tests in wet tuff consist of approximately 80 points (Reference 26, page 43). A linear regression analysis was performed with the logarithms of the peak velocities and corresponding normalized ranges. The resulting equation for the median of the applied free-field compressive stress in wet tuff is

$$(P_o')_{0.5} = (5.6 \times 10^6)(K_W \cdot W_S)^{0.52} (D^2 + LR^2)^{-0.78} \tag{50}$$

where

P_o' = free-field stress (psi) at depth
W_S = weapon yield (KT) at surface
K_W = weapon effectiveness factor
D = depth of facility in meters and
LR = lateral range in meters.

The equation of the upper 90% confidence limit is similarly given by

$$(P_o')_{0.9} = (14.4 \times 10^6)(K_W \cdot W_S)^{0.52} (D^2 + LR^2)^{-0.78} \qquad (51)$$

Since the normal distribution is assumed for the logarithms of the velocities and ranges in the confidence interval calculation of Reference 26, the coefficient (A_o) in Equation (42) of paragraph 5 may be considered a lognormal random variable if the exponents are assumed constant.

This latter assumption is justified by the fact that the coefficient of variation of the exponents is less than six percent according to the regression analysis (Reference 26, page 42). The mean and coefficient of variation of the coefficient (A_o) for wet tuff are determined as follows using the inverses of Equations (11) and (12):

$$\mu + 1.645\sigma = \ln(14.4 \times 10^6) = 16.4827$$

$$\mu = \ln(5.6 \times 10^6) = 15.5383$$

$$\sigma = 0.5741$$

$$V_A = \sqrt{e^{\sigma^2} - 1} = 0.625$$

$$\overline{A}_0 = e^{\mu}\left(\sqrt{1 + V^2}\right) = 6.60 \times 10^6 \text{ psi}$$

The measured data from 9 underground tests in hard rock consist of approximately 150 points (Reference 26, page 47). A regression fit of these data provided the following equation for the median of the free-field stress in hard rock:

$$(P_O')_{0.5} = 34.1 \times 10^6 \, (K_W \cdot W_S)^{0.5733} \, (D^2 + LR^2)^{-0.86} \qquad (52)$$

The corresponding equation for the upper 90% confidence limit is:

$$(P_O')_{0.9} = 93.3 \times 10^6 \, (K_W \cdot W_S)^{0.5733} \, (D^2 + \text{[illegible]})^{-0.86} \qquad (53)$$

If the exponents are assumed constant, the mean and coefficient of variation for the lognormal coefficient (A_o) for hard rock are $\overline{A}_o$ = 41.1 X 10^6 psi and V_A = 0.674. Since the coefficient of variation of the exponents is 7.3 percent, according to the regression analysis (Reference 26, page 45), this assumption is considered justified.

9.2 Geometrical and Material Property Data

The allowable strength corresponding to a specified circumferential strain at failure is determined from Newmark's equation using the geometrical and material parameters defined in Appendix A.1. Data values for these parameters are presented in Table 4. The data for the steel liner were obtained from the Wiggins study of the Mighty Epic test (Reference 8, page 3-15). The data for the concrete unconfined compressive strength, the concrete elastic modulus, and the Poisson's ratio for concrete were obtained from the 1971 ACI code (Reference 30). Dynamic factors to account for high strain rate effects are included in the steel and concrete

Table 4. Geometrical and material property data.

Parameter	Symbol	Unit	Mean (m)	Coefficient of Variation (V)
Steel outer radius	r_a	inch	84	0.005
Steel yield stress	σ_y	psi	52800	0.06
Steel liner thickness	h_s	inch	(Design variable)	0.003
Concrete unconfined compressive stress	f_c'	psi	7020	0.0556
Concrete friction-dependent constant	K_{sc}	--	4.1	0.341
Concrete elastic modulus	E_c	psi	5.0×10^6	0.04
Concrete Poisson's ratio	ν_c	--	0.30	0
Wet tuff unconfined compressive stress	σ_{ur}	psi	4000	0.475
Wet tuff friction-dependent constant	K_{sr}	--	1.19	0.17
Wet tuff elastic modulus	E_r	psi	0.653×10^6	0.306
Wet tuff Poisson's ratio	ν_r	--	0.39	0
Concrete outer radius	r_{co}	inch	(Design variable)	0.02
Granodiorite unconfined compressive stress	σ_{ur}	psi	24600	0.226
Granodiorite friction-dependent constant	K_{sr}	--	2.45	0.205
Granodiorite elastic modulus	E_r	psi	9.31×10^6	0.194
Granodiorite Poisson's ratio	ν_r	--	0.31	0

data as recommended by Reference 28. The data for the friction-dependent constant for concrete were determined from confined compressive test data (Reference 28, paragraph 8.2.2) as shown in Figure 8.

The rock data were based on existing data from two different underground test programs. The site location for the hypothetical soft rock design was assumed to be near the structures drift in the Mighty Epic nuclear test site. The rock data for this site are based on data from References 31 and 32. The results of a linear regression analysis using the compression test data are shown in Figure 9. The site location for the hypothetical hard rock design was assumed to be near the Hard Hat test site. The rock data for this site are based on data from Reference 33. Figure 10 illustrates the linear regression analysis performed with the compression test data.

Two basic assumptions were involved with using these rock strength properties. First, the effect of overburden or in situ stress on rock strength is neglected. The rock properties used for this study are based on bore hole data from depths not necessarily the same as the depth of the hypothetical facility. It was assumed that a strength increment existed which was above the rock strength used in the analysis and which was sufficient to react the static long-term overburden loads. Second, the measured rock strength properties are assumed to represent accurately the in situ properties of the rock. Thus local effects of jointing and inhomogeneities occurring in the in situ rock mass but not in the particular bore hole specimens are neglected. However, the granodiorite specimens tested and described in Reference 33 did contain small joints which significantly affected the measured rock properties. Also, the Mighty Epic bore hole specimens indicated a significant strength variation along the tunnel. Since this wet tuff was treated as a single rock geology, the variation along the tunnel was included in the statistical variations of the rock properties.

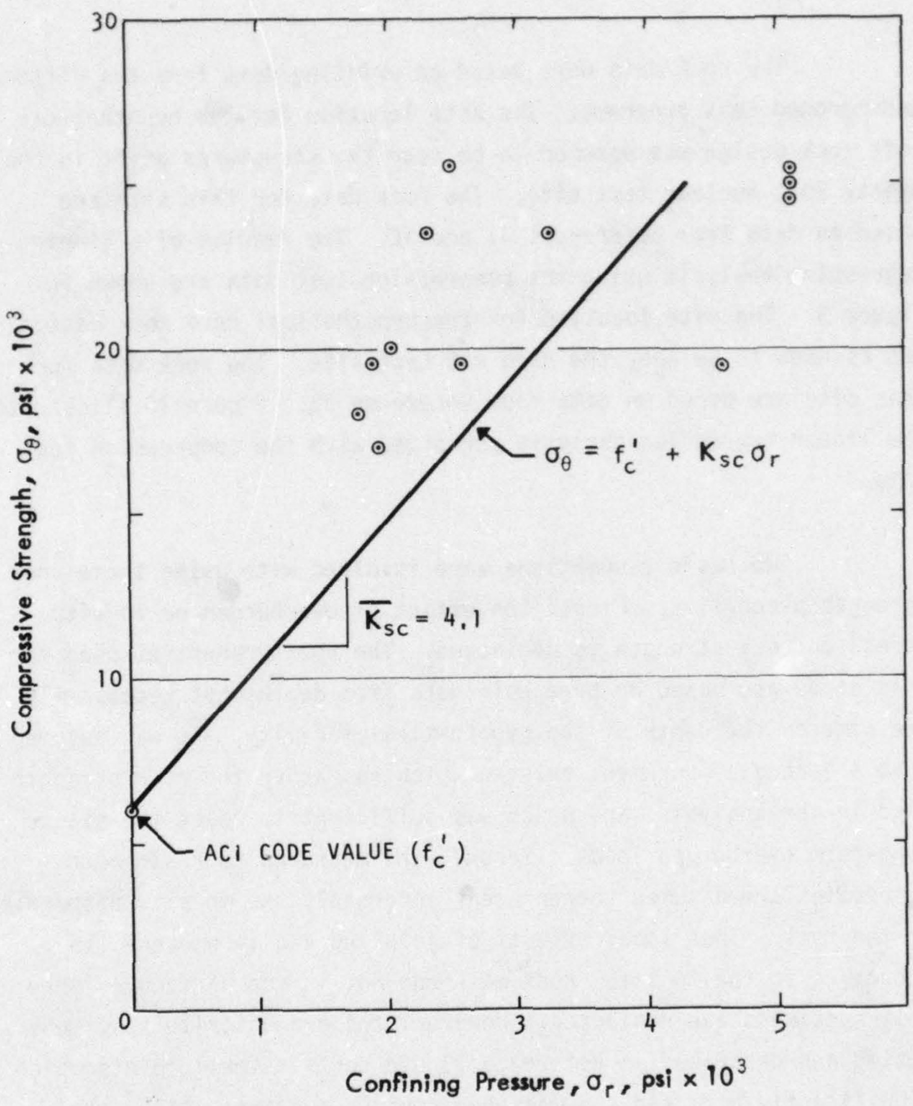


Figure 8. Concrete strength.

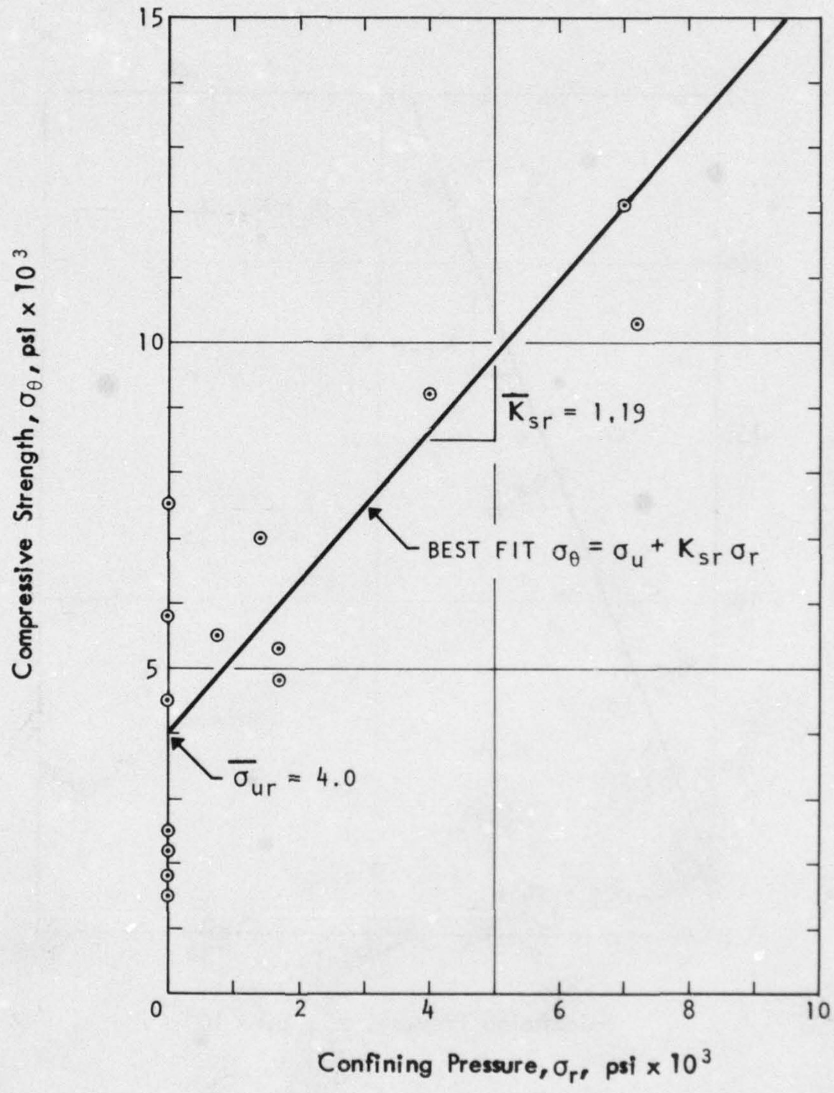


Figure 9. Strength of wet tuff.

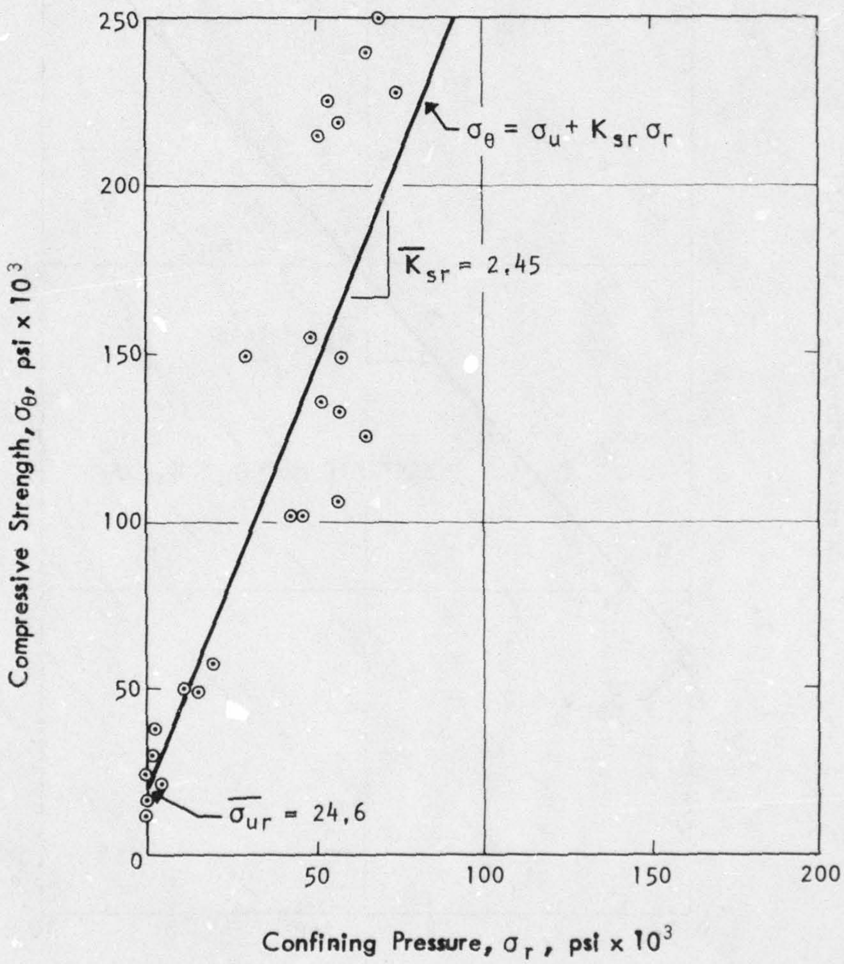


Figure 10. Strength of granodiorite.

The rock strength properties for a real design situation should be obtained from bore hole specimens from the depths of interest. This is because rock strength is affected by the depth and in situ stress at which the rock was formed. The required triaxial compression tests should duplicate as nearly as possible the principal stress relationships and strain rates occurring in the design load conditions. In particular, the test confining pressures (σ_r) should be within the range of interest for the design.

9.3 Failure Strain Data

According to Newmark's method, failure of the rock/liner system is assumed to occur when circumferential strains exceed allowable values. For this study, excessive strains in the concrete/steel liner (ε_c) and in the rock (ε_r) are both assumed to be possible independent failure modes. Structural failure is defined as the point at which unlimited strain occurs with any increase in load. This definition of structural failure is consistent with the functional failure of the system occurring when the liner impacts the contained equipment. This catastrophic motion could result either from liner buckling or from plastic shear flow of the rock causing local liner failure due to nonuniform loading.

The test data base for residual liner strains and interface pressures between the rock and concrete is based on the 60-day results from the Mighty Epic event (Reference 34). Since the measured liner strains at failure indicate significant bending, these strains were averaged around the circumference for compatibility with the assumption of axial symmetry required by Newmark's equation. The average strain for each liner specimen was computed using mean rock, concrete, and steel properties. These strains were compared with the test predictions, the conclusions of Reference 8, and the degree of failure in the test (complete, partial, or none). Based on these data and subjective considerations, the following values were assigned to the mean and coefficient of variation of the concrete/steel liner failure strain:

$m_{sc} = 0.04 \qquad V_{sc} = 0.59$

Excessive strain in the rock implies a sufficient volume of rock being in a state of plastic flow that bulking (expansion) along a flow line leads to a catastrophic nonuniform loading of the liner. Since an empirical data base is currently unavailable for this type of failure, the rock strains at failure were based on triaxial test results from References 32 and 33. For the wet tuff, the following values were assigned to the mean and coefficient of variation of failure strain:

$m_{sr} = 0.06 \qquad V_{sr} = 0.04$

The corresponding values for the granodiorite failure strain are as follows:

$m_{sr} = 0.05 \qquad V_{sr} = 0.55$

9.4 Cost Data

The cost data are based on data from the Hardrock nuclear test program, data provided informally by the DNA field command, data from the Deep Sanguine project, and 1976 Means Cost Data, Ref. 37. All cost data have been adjusted to a 1976 base. The data are used only to illustrate the effects of cost in the design of the hypothetical deep-buried facility. The general cost function proposed to estimate construction costs of deep-buried tunnels is given by Equation (48) of paragraph 8. The values of the cost parameters used for the present study are as follows:

MM = 305	F = 0.08
BB = .0	G = 2.33
C = 99.0	H = 0.0178
E = 0.202	

10. DETERMINISTIC DESIGN STUDIES

The deterministic design procedure is based on the Newmark method described in paragraph 8 and Appendix A.1 and on the data base described in paragraph 9. The designer first selects a trial steel and concrete liner section. The governing failure condition is then determined by substituting the rock mean failure strain (m_{sr}) for ε_r in Equation (54) and then comparing the resulting ε_c with the concrete mean failure strain (m_{sc}).

$$\epsilon_c = RCA^2 \cdot \epsilon_r + \frac{NC}{EC}\left(\sigma_{ra} - RCA^2 \cdot \sigma_{rc}\right) \tag{54}$$

Depending on which failure condition is critical, the allowable free-field stress is then determined from Equations (A-1) and(A-4) combined with either Equation (A-2) or Equation (A-3). The resulting allowable free-field stress (P_o) is then compared with the applied free-field pressure (P_o') from the Perret-Bass equations (paragraph 9.1) to determine the required burial depth. The construction cost for this design is then determined from Equation (48). This procedure may be repeated with different trial liner sections.

For the present deterministic design studies, the critical failure strains occur in the steel/concrete liner for both types of rock. Nominal (mean) values are used for failure strain and for all geometrical and material properties except for rock strengths. The rock strength properties are obtained as the "lower bounds" of the triaxial test data shown in Figures 9 and 10. The applied free-field pressure coefficients (A_o) are the upper 90 percent confidence limits. The construction costs are based on a tunnel length of 100 meters. These deterministic design data are included with a summary of the design calculations in Table 5.

Table 5. Deterministic design summary.

	Tuff	Granodiorite	Source
r_a (inch)	84	84	Paragraph 7
h_s (inch)	1.25	1.25	Trial Design Variable
σ_{ra} (psi)	785.7	785.7	Equation (A-10)
r_{co} (inch)	96	96	Trial Design Variable
σ_{rc} (psi)	2350	2350	Equation (A-11)
ε_c -	0.04	0.04	Equation (54)
σ_{ur} (psi)	2000	15400	Figures 9 and 10
K_{sr} -	1.3	1.6	"Lower Bound" Values
P_o (psi)	7032	46035	Equations (A-1), (A-3), (A-4)
A_o (psi)	14.4×10^6	93.3×10^6	Upper 90% Confidence Limit
K_w -	0.04	0.04	Paragraph 7
W_s (KT)	25000	25000	Paragraph 7
D (m)	1326	837	Equation (42)
CI (dollars)	4.42×10^6	2.11×10^6	Equation (48)

11. PROBABILISTIC DESIGN STUDIES

A probabilistic design approach for deep-buried lined tunnels requires descriptions of the randomly varying loads and strengths. The random applied loads are the lognormal free-field stresses from the Perret-Bass data (Reference 26). The random allowable strengths are obtained from the deterministic analytical relationships of Newmark's equation (paragraph 8 and Appendix A.1) by means of the Taylor's series method. As suggested by Equation (17), the mean of the allowable free-field stress is approximated by Equation (A-1) evaluated at the mean values of all the variables and parameters, including the circumferential failure strain. The corresponding variance of the allowable free-field stress, as suggested by Equation (18), is approximated in terms of the partial derivatives of Newmark's equation. The partial derivatives required for the rock failure mode and for the concrete/steel failure mode are presented, respectively, in Appendices A.2 and A.3.

Failure of the deep-buried tunnel is assumed to occur when circumferential stresses exceed allowable values either in the rock or in the steel/concrete liner. If the allowable rock strain and allowable liner strain are approximately equal, the failure probability may be determined from the assembly reliability approach of Equations (8) through (12). Here the lognormal applied load is the free-field compressive stress (P_o') given by the Perret-Bass relationship (Equation 42). The two allowable strengths are the free-field compressive stresses (P_o) corresponding to the rock and liner failure strains. The allowable strengths are also considered to be lognormally distributed by virtue of the central limit theorem (Equations 25 through 30). The allowable strengths are assumed to be statistically independent even though the functional relationship imposes a small positive correlation. This latter assumption therefore results in a slight conservatism in the calculated failure probability.

Figure 11 illustrates the significance of considering explicitly the two possible failure modes in a probabilistic analysis. When either of the two failure modes could reasonably occur, then the assembly reliability approach is recommended. When the mean failure strain for the rock greatly exceeds that for the liner, only the liner failure mode need be considered. The converse is true, of course, when the mean liner strain greatly exceeds that for the rock. When only one of the two failure modes could reasonably occur, then the component reliability approach of Equations (1) through (6) is recommended. If median values of the lognormal load and strength are used to define the design limit load and allowable strength, Equation (5) reduces to the following general equation for component probability of failure:

$$F^{-1}(P_F) = \frac{\ln u + \mu_L - \mu_S}{\sqrt{\ln\left[(1+V_L^2)(1+V_S^2)\right]}} \qquad (55)$$

where

μ_L and μ_S are the transformed normal parameters given by Equation (11).

The probabilistic design procedures is directly analogous to the deterministic design procedure described in paragraph 10 except that the failure probability of the rock/liner system is explicitly considered. The procedure is based on the Newmark method (Appendix A) and on applicable probabilistic concepts (paragraph 2). The procedure also incorporates all the information included in the data base described in paragraph 9. As with the deterministic procedure, the designer first selects a trial steel and concrete liner section. The governing failure condition is then studied using Equation (54) of paragraph 10. If either of the two failure modes can be ignored, the required design limit load (L_D) is determined from Equation (5) as the

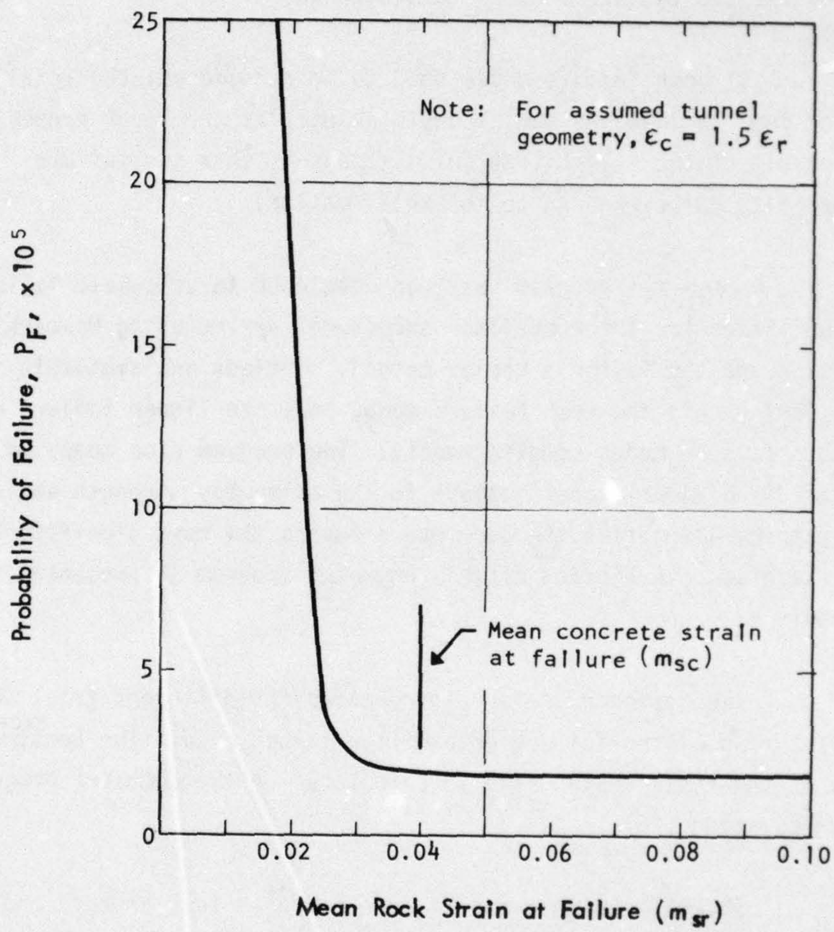


Figure 11. Assembly failure probability for deep-buried tunnel.

allowable strength (S_A) divided by the factor of safety (FS) for a specified probability of failure (P_F). The corresponding depth can then be determined given the design limit load and the Perret-Bass equations (paragraph 9.1). The construction cost for this trial design is again determined from Equation (48). This procedure is directly analogous to the deterministic design procedure so long as one of the two failure modes is disregarded.

If both failure modes need to be considered, the trial design must include the burial depth as well as the liner properties. The result of the reliability calculations is then the failure probability corresponding to the trial design.

A computer program has been developed to calculate failure probabilities for the rock/liner structural system using Newmark's equation and the Taylor's series method. Options are available for considering only the rock failure mode, only the linear failure mode, or both failure modes simultaneously. The program also computes the required individual contributions to the allowable strength variance and thereby identifies the parameters having the most significant uncertainties. A listing of this computer program is presented in Appendix B.

The design procedure is repeated for different trial designs until the specified failure probability is achieved. The construction cost of the final design is also calculated by the computer program using Equation (48).

Several design examples were computed for the tuff and granodiorite geologies from the data base described in paragraph 9. The computer program was used to calculate failure probability and cost corresponding to fixed liner properties at several different

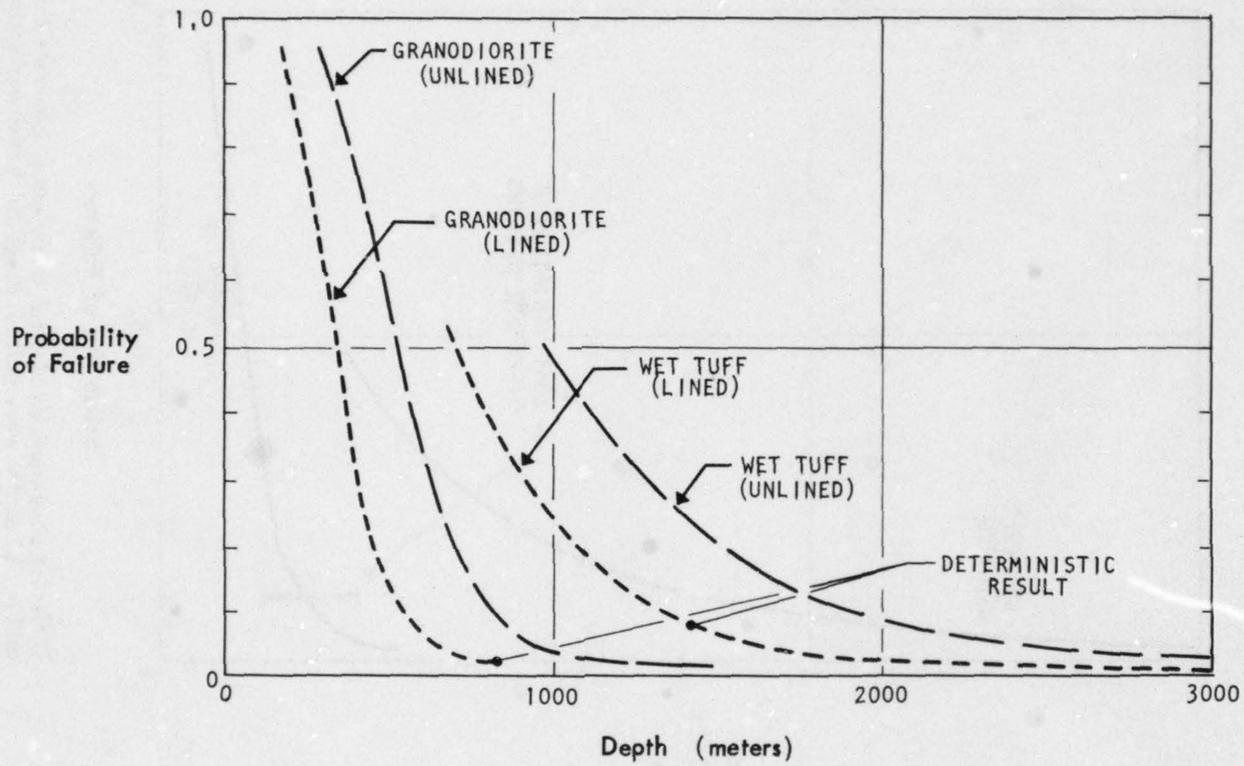
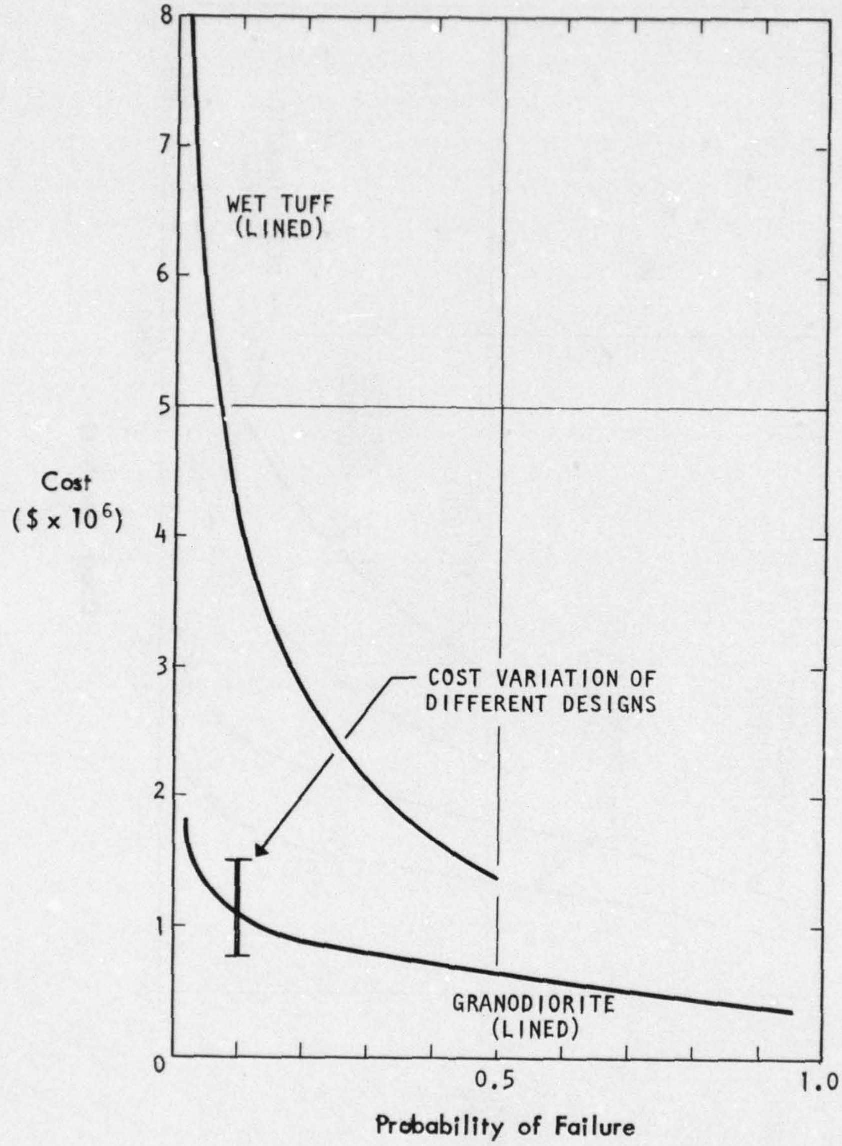


Figure 12. Probability of failure versus depth of burial.



Different combinations of steel thicknesses, concrete thickness, and burial depths were varied. Steel thickness was varied from 6.35 mm to 63.5 mm. Concrete thickness was varied from 0 to 381 mm. Burial depths were varied from 550 mm to 820 m. Steel, concrete and rock strengths were not varied.

Figure 13. Cost versus probability of failure.

depths of burial. The results for typical costs are shown in Figure 12 as failure probability versus depth both for lined tunnels and for unlined tunnels. Several designs were based on the deterministic approach in paragraph 10. The probability failure of each design was computed and is shown in Figure 12. Failure probabilities for designs based on the deterministic approach (paragraph 10) were also calculated and are also shown in Figure 12. As indicated, the deterministic design approach results in failure probabilities which are neither clearly specified nor the same. The probabilistic design approach uses the data base in a complete and rational manner and provides a direct assessment of the failure probability.

The probabilistic approach also permits meaningful studies of failure probability versus cost as shown in Figure 13 for lined tunnels. Such data does provide an approach to decision making regarding the economy of dispersion. For example, when the cost of decreasing the failure probability from P_F to $(P_F)^2$ exceeds 100 percent, increasing the number of facilities from one to two is economical.

Figure 13 also indicates the economy of optimizing the design parameters to minimize cost for a specified failure probability. As shown for the granodiorite example, the range of cost variations for different designs may encompass a factor of two for a given failure probability. The uncertainty in the rock properties affects the cost comparisons between wet tuff and granodiorite. Examples in tuff show an even greater cost variation for small failure probabilities because of the greater uncertainty in the tuff load and strength properties as compared with the granodiorite properties.

The Taylor's series approach provides a rational way of identifying the design parameters whose uncertainties have the greatest impact on the design cost. The critical parameters in order of importance are as follows:

(1) applied free-field stress at depth (P_o')
(2) rock strength (K_{sr} and σ_{ur})
(3) failure strain (ε_r and ε_c)
(4) concrete strength (K_{sc} and f_c').

The elastic properties of the rock and concrete do not have a significant effect on the rock/liner system design and its resulting cost.

All the estimated construction costs have been for very short tunnel lengths. A study was performed to determine the effect of tunnel length in a practical design situation by comparing costs for a tunnel having either no liner or an inexpensive commercial liner. The patented Bernold liner consists of perforated sheet metal anchored to the rock with shotcrete placed between sheet metal and rock. For any given failure probability, the lined tunnel requires a shallower burial than the unlined tunnel. Figure 14 shows the tunnel length at which the lined tunnel construction cost equals the unlined tunnel cost as a function of failure probability.

The coefficient of uncertainty (u) is defined in paragraph 2 to account, in a conservative manner, for uncertainties in the analytical prediction of the random loads and strengths. For practical designs, this coefficient may vary from 1.1 for sophisticated analytical methods to perhaps 1.5 for less detailed methods. For convenience in this design example, the coefficients of uncertainty for load and strength were assigned the value of unity for all cases.

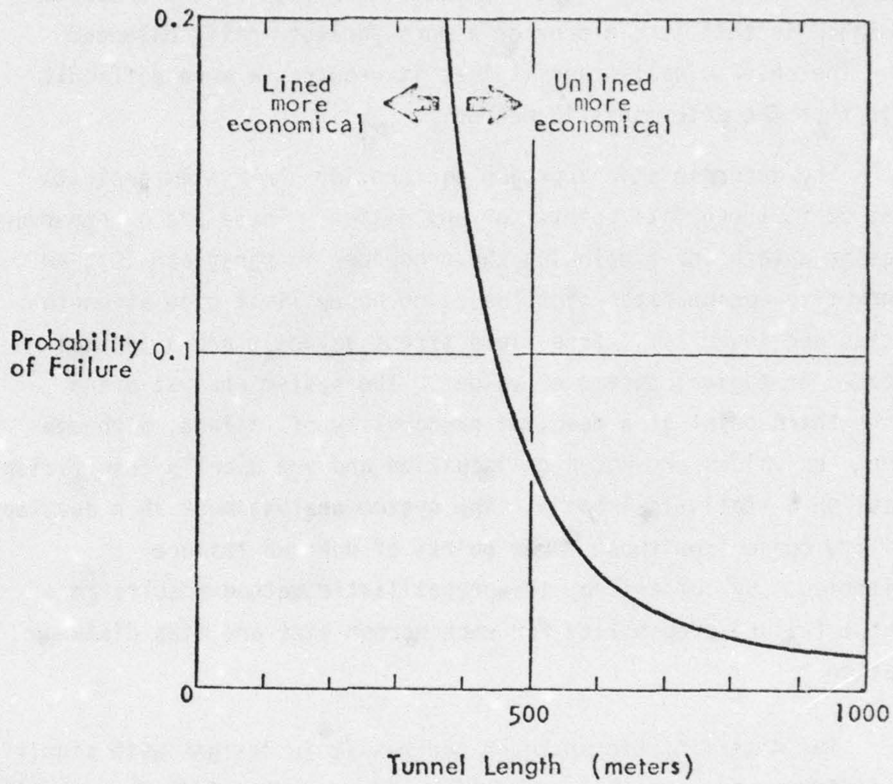


Figure 14 Economy of tunnel lining versus length

12. EVALUATION OF PROBABILISTIC STRUCTURAL DESIGN APPROACH

The probabilistic method as compared to the deterministic method has several advantages and disadvantages when applied to hardened structures. Generally, the chief advantage of the probabilistic method is that it can provide a more parametrically balanced design. The chief disadvantage is that it requires a more difficult analysis than the deterministic method.

The deterministic approach can provide the system analysts with, at best, three data points for any design. These are a conservative design data point (following the procedure in paragraph 10), an unconservative design data point (based on upper limit code strength properties and lower limit free-field stress values), and a typical value based on typical parameter values. The system analyst often takes the third point at a mean, at probability of failure, although the parameter values are based on intuition and are usually insufficient to establish a statistical basis. The system analyst must then develop a fragility curve from these three points of unknown failure probabilities. By comparison, the probabilistic method results in a meaningful failure probability for each weapon size and miss distance combination.

The deterministic approach can result in designs with significantly different degrees of conservatism, for example, failure probabilities corresponding to the two deterministic designs are shown in Figure 12 of paragraph 11. From the same conservative approach, failure probabilities of 0.02 and 0.09 resulted for the designs in granodiorite and wet tuff, respectively. This lack of a consistent standard of comparison could obviously lead to incorrect conclusions in a design trade study. If a failure probability larger than those corresponding to the deterministic approach had been rationally selected as the specified design value, and if the design had been conducted using probabilistic methods, savings in construction costs and in excavation time could have resulted. However, if a failure probability smaller than those corresponding to the deterministic approach had been selected as the specified design value, the "conservative" deterministic designs would have, in fact, been unconservative.

The chief difficulties in the probabilistic method are the statistical calculations required to establish the means and coefficients of variation for the various strength parameters by quantification of uncertainties, and the numerical calculations required to determine failure probabilities.

SECTION IV
STRUCTURAL TESTING APPROACH

13. TEST PHILOSOPHIES AND TECHNIQUES

Several tests may be appropriate to support analytical design studies of deep-buried rock/liner structural systems. These tests can be classified into structural element tests and structural system tests for determining allowable strength, and geological tests for determining applied free-field stress.

Structural element tests are performed to evaluate the required material mechanical properties. Examples of this type of test include the following:

(1) steel tensile coupon tests to obtain the yield stress (σ_y);
(2) concrete unconfined and confined compressive tests to obtain strength properties (f'_c and K_{sc}) and elastic properties (E_c and ν_c);
(3) rock unconfined and confined compressive tests to obtain strength properties (σ_{ur} and K_{sr}) and elastic properties (E_r and ν_r).

These properties are consistent with the particular assumptions of the Newmark method (paragraph 8). If the Mohr-Coulomb failure condition is not assumed for the rock and concrete, the compressive test data may be interpreted to fit a nonlinear idealization. For the triaxial compressive tests, the confining pressures should approximate the design applications. For all the structural element tests, the applied strain rate should be approximately the same as that implied by the design load conditions. A sufficient number of these small sample tests should be performed for a statistical evaluation of the material properties.

An important problem related to rock mechanics is determining the number of test data points required to describe the properties of a particular rock layer. If the rock layer is well identified and relatively homogeneous, an estimate of required sample size may be obtained from nonparametric sampling theory. According to Reference 35, the probability (p) that at most q x 100 percent of the future tests will have values less than the smallest value in n tests is given by:

$$p = 1-(1-q)^n \tag{56}$$

For the homogeneous rock layer, q x 100 may be interpreted as the maximum percentage of the total rock volume having material properties less than those measured in n nominally identical tests of samples randomly selected from the rock volume. The probability (p) is the confidence level. As a numerical example, if $p = 0.9$ and $q = 0.1$, the required number of tests is 22. No assumptions regarding the underlying probability distributions of the material properties are required for this approach.

The second class of tests appropriate to deep-buried facilities includes tests of the structural system. These tests are performed primarily to increase understanding of the structural behavior of the rock/liner system. Examples of this type of test include the following:

(1) uniform and nonuniform compressive tests of the composite steel/concrete liner;
(2) structural/medium interaction tests involving both rock and composite steel/concrete liner.

These tests may be either full-scale or reduced-scale model tests. The waveform of the applied stress wave implied by the design load conditions should be simulated as accurately as possible

in the tests. Since the cost of this type of test would limit the number of samples available, the concepts of Bayesian statistics would be appropriate for the interpretation of the test results (see, for example, Reference 16).

The third class of tests includes tests of the geology to evaluate wave-transmission characteristics required for free-field stress calculations. Examples of this type of tests include the following:

(1) rock triaxial compressive tests to determine material properties;
(2) uni-axial dynamic strain ("flyer-plate") tests to determine the relationship between wave speed and free-field stress;
(3) in situ tests to relate the actual material properties of the rock volume in its natural position to the results of tests performed with bore-hole samples.

A sufficient number of the triaxial compression and "flyer-plate" tests should be performed for a statistical evaluation of the pertinent rock properties.

The analytical design studies appropriate for deep-buried rock/liner structural systems involve either the allowable strength or the applied free-field compressive stress. The allowable strength may be determined from finite-element structure/media interaction calculations instead of by the Newmark method discussed in paragraph 8. A nonlinear structural analysis computer program, such as that described in Reference 36, provides more realistic idealizations of the appropriate stresses and strains in terms of the steel, concrete, and rock material properties. The applied free-field stress may be calculated between the weapon impact point and the deep-buried facility. The resulting free-field stress calculation is based on a

specific geology rather than on the variety of geologies implied by the Perret-Bass data (Reference 26). This calculation, which would provide an accurate free-field stress at depth, would involve weapon coupling with the free surface, crater effects, and stress transmission through the rock.

14. APPLICATION TO EXAMPLE HARDENED FACILITY ELEMENT

The selection of the optimum test program for a deep-buried rock/liner structural system may be rationally approached with the concepts of Bayesian statistical decision theory (paragraph 3). To illustrate the rational selection of a test program, consider the following example based on the numerical studies of paragraph 11. A deep-buried facility is to be constructed at a site having granodiorite below a depth of 400 meters but softer rock above 400 meters. The design probability of failure has been specified as 0.1 percent. A design has been completed with the Newmark method used for allowable strength and with the Perret-Bass data used for applied free-field stress. The liner has been optimized for the granodiorite rock properties. The softer rock layers above 400 meters are believed to attenuate the free-field stress somewhat more than indicated by the Perret-Bass data. Design studies with the Taylor's series method indicate that the uncertainty in applied free-field stress (P_o') has the greatest significance on the predicted failure probability. Therefore, a reduction in the coefficient of variation of P_o', which is expected from a free-field stress calculation using specific local geological properties, could result in significant construction cost savings. Furthermore, the free-field stress calculation could account for the potentially beneficial effects of the softer rock layer above 400 meters. The two optional actions are directly related to the outcomes of the following experiments:

- e_o: deploy the existing design based on $P_F = 0.001$ and the Perret-Bass data or
- e_1: delay deployment and redesign the facility based on $P_F = 0.001$ and the results of a free-field stress calculation and the supporting rock triaxial compression tests, "flyer plate" tests, and in situ tests.

The results of the tests and analysis are, of course, unknown "a priori." Nevertheless reasonable test/analysis outcomes may be hypothesized and a subjective probability of occurrence may be assigned to each outcome. Relative to the Perret-Bass data, the mean analytical free-field stress (A_0) is more likely to decrease than to increase because of the attenuating effects of the soft rock layer. The coefficient of variation of the calculated stress, (V_A), which is based on local geological properties, is much more likely to decrease than to remain the same. Values assigned to the mean stresses are assumed statistically independent of the values assigned to the coefficients of variation. Therefore the following experimental outcomes (Z_i) and assigned probabilities (P_i) are considered reasonable:

Z_1	$A_0 = 30 \times 10^6$ psi	$V_A = 0.674$	$P_1 = .15$
Z_2	$A_0 = 30 \times 10^6$ psi	$V_A = 0.5$	$P_2 = .35$
Z_3	$A_0 = 41.1 \times 10^6$ psi	$V_A = 0.674$	$P_3 = .09$
Z_4	$A_0 = 41.1 \times 10^6$ psi	$V_A = 0.5$	$P_4 = .21$
Z_5	$A_0 = 50 \times 10^6$ psi	$V_A = 0.674$	$P_5 = .06$
Z_6	$A_0 = 50 \times 10^6$ psi	$V_A = 0.5$	$P_6 = .14$

Each of the actions corresponds directly to one of the test/analysis outcomes, since the action is merely to redesign the facility according to the particular outcome. The decision logic is therefore truncated with the payoff effectively occurring at the test/analysis outcomes. The cost of each action is the construction cost (Equation 48) plus, where applicable, the following additional cost of the stress calculations, the supporting tests, and the delay in deployment:

(a)	additional rock samples	\$50,000
(b)	additional laboratory tests	\$30,000
(c)	in situ tests	\$40,000
(d)	free-field stress calculations	\$100,000
(e)	delay in deployment of 20 days @ \$10,000/day	\$200,000
		\$420,000

The truncated decision tree describing this hypothetical situation is shown in Figure 15. The expected value of option e_o is 4.0×10^6 dollars while the expected value of option e_1 is 3.36×10^6 dollars. Therefore, based on the postulated experimental outcomes and assigned probabilities, the decision to delay deployment and redesign the facility is highly favored. In addition, the amount by which the analysis and test option is favored indicates that considerably more resources could be expended for the test/analysis effort than was initially planned, or an additional delay in deployment could be tolerated.

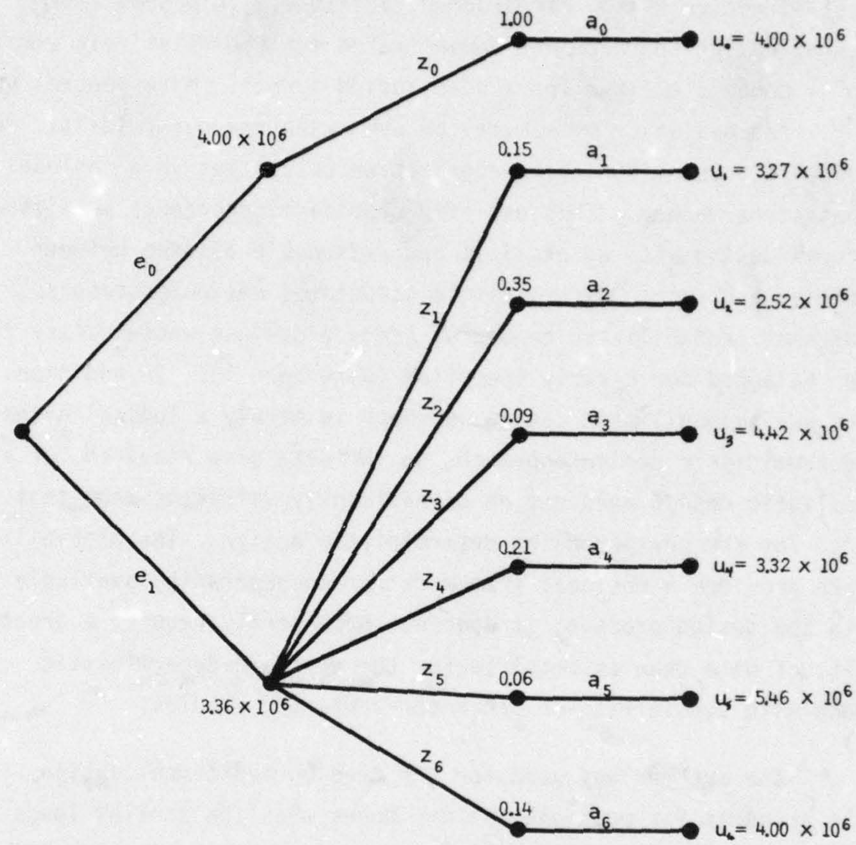


Figure 15. Truncated decision tree for selection of test program.

SECTION V
CONCLUSIONS

The fundamental conclusion of the present study is that the methodology is now available to use probabilistic concepts as the basis for cost-effective design of hardened facilities. A probabilistic structural design approach was demonstrated for the relatively simple case of a composite liner for a deep-buried tunnel. More generally, however, probabilistic conceptsmay be used with any deterministic design procedure to account for the recognized uncertainties in a rational and consistent manner. This use of probabilistic concepts permits a structural design with an explicit and reasonable balance between reliability and cost. Deterministic structural design procedures, used without probabilistic concepts, produce designs whose safety is neither balanced nor clearly specified (Reference 1). In addition, just as any probabilistic design approach is merely a logical extension of a deterministic design approach, so the data base required for a probabilistic design need not be significantly different from that required for the corresponding deterministic design. The probabilistic approach provides a rational framework for incorporating available data in the design process; it does not necessarily require a greater quantity of data than is required for the standard deterministic approach with sensitivity or parameter-variation studies.

The methodology used for the deep-buried tunnel design example accounts for multiple failure modes when the applied loads result from a single source such as a nuclear event. Although both loads and strengths are required to follow either the lognormal or the normal probability law and the strengths are required to be independent, this basic approach is believed to have wide applicability in the structural design of hardened facilities.

Basic concepts of Bayesian statistical decision theory are presented along with two practical examples. The first example (paragraph 4) results in a method for rationally selecting an acceptable failure probability for hardened facilities. The second

example (paragraph 14) involves the use of decision theory in defining structural test programs. Bayesian statistical decision theory is a powerful tool to balance risk against cost for decision-making under uncertainty.

The existing data base (paragraph 9) was adequate to provide realistic numerical values for use in the example design. The applied load ($P_o{}'$) data and the geometrical and material property data and the failure strain data for allowable strength (P_o) are typical of currently available values.

SECTION VI
RECOMMENDATIONS

Future research areas with general applicability to hardened facilities include the following:

1. Develop and demonstrate detailed procedures for applying the minimum expected cost hypothesis to determine acceptable failure probabilities of hardened facilities. This involves defining specific failure conditions for hypothetical hardened facilities, preparing and distributing appropriate questionnaires to members of the defense agencies, and interpreting the results.

2. Apply statistical decision theory to specific decisions affecting the actual design and test of hardened facilities. Application to an existing design or test is necessary for the purpose of demonstrating the advantages and/or disadvantages of the method when compared to existing design and test procedures.

3. Extend the probabilistic structural design approach by a general incorporation of finite-element analytical techniques.

The study of deep-buried lined tunnels in this report suggests the following areas of future research in the design of rock/liner systems.

1. Develop and demonstrate methodology for minimizing total cost subject to a specified failure probability for a given site using the existing design equations based on Newmark's method. Design variables to be included in the optimization procedure are depth of burial, dimensions of the steel/concrete composite liner, and concrete strength.

2. Develop probabilistic design equations, for the rock/liner structural system, which are not limited by the assumption of radial symmetry. This involves incorporating the effects of local variations in applied load and in rock strength around the perimeter of the tunnel as in Reference 29.

3. Develop and demonstrate methodology for applying finite-element techniques to the probabilistic design of rock/liner systems. In particular, application of the NONSAP code (Reference 36) will be studied.

SECTION VII

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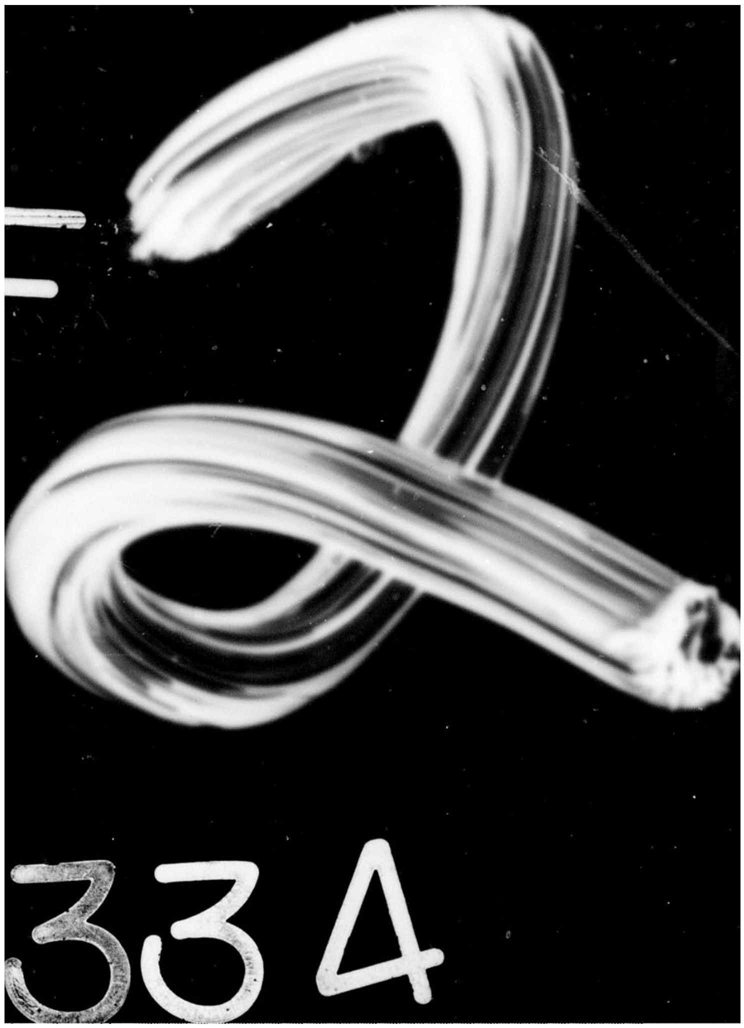
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APPENDIX A
DESIGN EQUATIONS BASED ON
NEWMARK'S METHOD

A.1 NEWMARK'S EQUATION

The free-field compressive stress, expressed in terms of geometric and material properties of the rock/liner structural system, is determined as follows from Newmark's method when the elastic/plastic interface is in the rock:

$$P_o = \frac{(K_{sr}+1)}{2(K_{sr}-1)} \left[\{S_{ri}\}^{\left(\frac{K_{sr}-1}{K_{sr}+1}\right)} \bullet \{Y_{ri}\}^{\left(\frac{2}{K_{sr}+1}\right)} \right] - \left(\frac{\sigma_{ur}}{K_{sr}-1}\right) \tag{A-1}$$

where $S_{ri} = ER \bullet \epsilon_r - NR \bullet \sigma_{rc}$ for the rock failure condition, (A-2)

$$S_{ri} = \frac{ER}{EC} \left[\left(\frac{EC \bullet \epsilon_c - NC \bullet \sigma_{ra}}{RCA^2} \right) + NC \bullet \sigma_{rc} \right] - NR \bullet \sigma_{rc} \tag{A-3}$$

for the concrete/steel failure condition, and

$$Y_{ri} = \sigma_{ur} + (K_{sr} - 1) \bullet \sigma_{rc} \tag{A-4}$$

The critical stresses and strains used in the Newmark formulation are as follows:

σ_{ra} ~ radial stress at concrete/steel interface applied by steel to concrete,

ϵ_c ~ circumferential strain at concrete/steel interface for concrete/steel failure condition,

σ_{rc} ~ radial stress at rock/concrete interface applied by concrete to rock, and

ϵ_r ~ circumferential strain at rock/concrete interface for rock failure condition.

The free-field stress (P_o) corresponding to ϵ_c or ϵ_r is the allowable strength for the concrete/steel failure mode or the rock failure mode, respectively. The cumulative distribution function of the allowable strength (P_o) for a specific failure mode is usually referred to as the "fragility curve".

The following equations define the combination variables and parameters used in Equations (A-1) through (A-4) in terms of the basic quantities normally used with Newmark's method. These combinations are used only to simplify this probabilistic study and are not necessary conditions for the probabilistic design approach.

$$EC = \frac{E_c}{(1-\nu_c^2)} \tag{A-5}$$

$$ER = \frac{E_r}{(1-\nu_r^2)} \tag{A-6}$$

$$NC = \frac{(1-2\nu_c)}{(1-\nu_c)} \tag{A-7}$$

$$NR = \frac{(1-2\nu_r)}{(1-\nu_r)} \tag{A-8}$$

$$RCA = \frac{r_{co}}{r_a} \tag{A-9}$$

$$\sigma_{ra} = \frac{hs \cdot fy}{r_a} \tag{A-10}$$

$$\sigma_{rc} = \frac{\left[\left(f_c' + (K_{sc}-1)\cdot\sigma_{ra}\right)\cdot RCA^{(K_{sc}-1)} - f_c'\right]}{(K_{sc}-1)} \tag{A-11}$$

The basic parameters normally used with Newmark's method are defined as follows:

σ_{ur} ~ rock unconfined compressive strength (psi)
K_{sr} ~ dimensionless friction-dependent constant for rock
E_r ~ rock equivalent elastic modulus (psi)
ν_r ~ dimensionless Poisson's ratio for rock
r_{co} ~ radial distance to rock/concrete interface (inches)
f'_c ~ concrete unconfined compressive strength (psi)
K_{sc} ~ dimensionless friction-dependent constant for concrete
E_c ~ concrete equivalent elastic modulus (psi)
ν_c ~ dimensionless Poisson's ratio for concrete
r_a ~ radial distance to concrete/steel interface (inches)
f_y ~ yield stress of steel liner (psi)
h_s ~ thickness of steel liner (inches)

A.2 PARTIAL DERIVATIVES OF NEWMARK'S EQUATION FOR THE ROCK FAILURE MODE

The free-field compressive stress (P_o) defined by Equations (A-1), (A-2), and (A-4) is the allowable strength corresponding to the rock failure strain (ϵ_r). For the probabilistic design study, the Taylor's series method is used to obtain approximate values for the mean and variance of P_o. The mean of P_o is, according to Equation (17) of paragraph 2, approximately equal to the expression for P_o evaluated at the mean values of all individual variables and parameters. The variance of P_o is, according to Equation (18), approximated by summing the products of the squared partial derivatives (evaluated at the individual mean values) times the individual variances. Partial derivatives of P_o were determined with respect to the following variables and parameters: K_{sr}, σ_{ur}, ER, ϵ_r, NR, and σ_{rc}. For convenience in the probabilistic design study, these six quantities were considered statistically independent. The partial derivatives are as follows:

$$\frac{\partial P_o}{\partial K_{sr}} = \frac{\sigma_{ur}}{(K_{sr}-1)^2} + \frac{2P_o}{(K_{sr}+1)}\left[\left(\frac{1}{K_{sr}+1}\right) \ln\,(ER \bullet \epsilon_r - NR \bullet \sigma_{rc}) + \frac{1}{\sigma_{ur}+(K_{sr}-1)} - \frac{1}{(K_{sr}-1)} - \left(\frac{1}{K_{sr}+1}\right) \ln\left[\sigma_{ur}+(K_{sr}-1)\right]\right] \quad \text{(A-12)}$$

$$\frac{\partial P_o}{\partial \sigma_{ur}} = \frac{1}{(K_{sr}-1)}\left[\left(\frac{2}{K_{sr}+1}\right)\left[\sigma_{ur}+(K_{sr}-1)\right]^{\left(\frac{1-K_{sr}}{1+K_{sr}}\right)} (ER \bullet \epsilon_r - NR \bullet \sigma_{rc})^{\left(\frac{K_{sr}-1}{K_{sr}+1}\right)} - 1\right] \quad \text{(A-13)}$$

$$Q_r = \frac{1}{2}\left[\frac{\sigma_{ur} + (K_{sr} - 1)\sigma_{rc}}{ER \bullet \epsilon_r - NR \bullet \sigma_{rc}}\right]^{\left(\frac{2}{K_{sr}+1}\right)} \tag{A-14}$$

$$\frac{\partial P_o}{\partial ER} = Q_r \bullet \epsilon_r \tag{A-15}$$

$$\frac{\partial P_o}{\partial \epsilon_r} = Q_r \bullet ER \tag{A-16}$$

$$\frac{\partial P_o}{\partial NR} = -Q_r \bullet \sigma_{rc} \tag{A-17}$$

$$\frac{\partial P_o}{\partial \sigma_{rc}} = -Q_r \bullet NR \tag{A-18}$$

A.3 PARTIAL DERIVATIVES OF NEWMARK'S EQUATION FOR THE CONCRETE/STEEL FAILURE MODE

The free-field compressive stress (P_o) defined by Equations (A-1), (A-3), and (A-4) is the allowable strength corresponding to the concrete/steel failure strain (ϵ_c). The variance of P_o is approximated by the Taylor's series method (Equation 18 of paragraph 2) in terms of the following variables and parameters: K_{sr}, σ_{ur}, ER, NR, σ_{rc}, EC, ϵ_c, NC, σ_{ra}, and RCA. The partial derivatives of P_o taken with respect to these 10 quantities (which for convenience are considered statistically independent) are as follows:

$$S_{ri} = \frac{ER}{EC}\left[\frac{(EC \bullet \epsilon_c - NC \bullet \sigma_{ra})}{RCA^2} + NC \bullet \sigma_{rc}\right] - NR \bullet \sigma_{rc} \qquad \text{(A-3)}$$

$$Y_{ri} = \sigma_{ur} + (K_{sr} - 1) \bullet \sigma_{rc} \qquad \text{(A-4)}$$

$$Q_c = \frac{1}{2}\left(\frac{Y_{ri}}{S_{ri}}\right)^{\left(\frac{2}{K_{sr}+1}\right)} \qquad \text{(A-19)}$$

$$\frac{\partial P_o}{\partial K_{sr}} = \frac{S_{ri}^{\left(\frac{K_{sr}-1}{K_{sr}+1}\right)}}{(K_{sr}-1)}\left[Y_{ri}^{\left(\frac{2}{K_{sr}+1}\right)}\left(\frac{\ln\left(\frac{S_{ri}}{Y_{ri}}\right)}{(K_{sr}+1)} - \frac{1}{(K_{sr}-1)}\right)\right.$$

$$\left. \bullet\, Y_{ri}^{\left(\frac{1-K_{sr}}{1+K_{sr}}\right)} \bullet \sigma_{rc}\right] + \frac{\sigma_{ur}}{(K_{sr}-1)^2} \qquad \text{(A-20)}$$

$$\frac{\partial P_o}{\partial \sigma_{ur}} = \frac{1}{(K_{sr}-1)}\left[S_{ri}^{\left(\frac{K_{sr}-1}{K_{sr}+1}\right)} \cdot Y_{ri}^{\left(\frac{1-K_{sr}}{1+K_{sr}}\right)} - 1\right] \qquad \text{(A-21)}$$

$$\frac{\partial P_o}{\partial ER} = \frac{Q_c}{EC}\left[\frac{(EC \cdot \epsilon_c - NC \cdot \sigma_{ra})}{RCA^2} + NC \cdot \sigma_{rc}\right] \qquad \text{(A-22)}$$

$$\frac{\partial P_o}{\partial NR} = -Q_c \cdot \sigma_{rc} \qquad \text{(A-23)}$$

$$\frac{\partial P_o}{\partial \sigma_{rc}} = Q_c\left(\frac{ER}{EC} \cdot NC - NR\right) + Y_{ri}^{\left(\frac{1-K_{sr}}{1+K_{sr}}\right)} \cdot S_{ri}^{\left(\frac{K_{sr}-1}{K_{sr}+1}\right)} \qquad \text{(A-24)}$$

$$\frac{\partial P_o}{\partial EC} = \frac{Q_c \cdot ER \cdot NC}{EC^2}\left(\frac{\sigma_{ra}}{RCA^2} - \sigma_{rc}\right) \qquad \text{(A-25)}$$

$$\frac{\partial P_o}{\partial \epsilon_c} = Q_c\left(\frac{ER}{RCA^2}\right) \qquad \text{(A-26)}$$

$$\frac{\partial P_o}{\partial NC} = Q_c\left(\frac{ER}{EC}\right)\left(\frac{-\sigma_{ra}}{RCA^2} + \sigma_{rc}\right) \qquad \text{(A-27)}$$

$$\frac{\partial P_o}{\partial \sigma_{ra}} = -Q_c\left(\frac{ER}{EC}\right)\left(\frac{NC}{RCA^2}\right) \qquad \text{(A-28)}$$

$$\frac{\partial P_o}{\partial RCA} = -2Q_c\left(\frac{ER}{EC}\right)\left[\frac{(EC \cdot \epsilon_c - NC \cdot \sigma_{ra}}{RCA^3}\right] \qquad \text{(A-29)}$$

APPENDIX B
COMPUTER PROGRAM FOR RELIABILITY OF DEEP-BURIED LINED TUNNEL

B.1 GENERAL

The computer program used for the probabilistic design study is described herein. Following a general description of the program and a definition of the required input data, are a complete program listing and printout for a sample problem. The computer program is written in FORTRAN IV for the CDC 6600.

B.2 PROGRAM DESCRIPTION

This program calculates the probability of failure and the initial cost of a deep-buried tunnel having a composite steel/concrete liner. Newmark's method (described in paragraph 8 and Appendix A) is used to relate material stresses and strains to applied free-field pressure. Failure of the rock/liner system is assumed to occur when circumferential strains exceed allowable values. Three optional descriptions of failure are available to the user as follows:

Option 0 - failure of rock at rock/concrete interface is single failure mode

Option 1 - failure of steel/concrete liner is single failure mode

Option 2 - failure of either rock or steel/concrete liner are possible independent failure modes.

B.3 PROGRAM INPUT, PROGRAM LISTING, AND SAMPLE PRINTOUT

PROGRAM CONTROL

OPTION	Option 0 (rock failure mode), option 1 (steel/concrete liner failure mode), option 2 (2 failure modes)
ND	Number of depths
DFL	Ratio of Minimum Depth/Mean Depth, $\overline{D}$
DFH	Ratio of Maximum Depth/Mean Depth, $\overline{D}$

INITIAL COST FUNCTION DATA (C_I)

R	Length of Tunnel (m)
MM, BB	Vertical Excavation Cost Data
C	Venting Cost Data
E, F	Horizontal Excavation Cost Data
G	Steel Liner Cost Data
HH	Concrete Liner Cost Data

APPLIED LOAD DATA - P_O Free Field Compressive Stress (mean and coefficient of variation)

$\overline{A}_O$ V_A	Rock Transmissibility (P_O coefficient)
$\overline{K}_W$ V_{Kw}	Weapon Effectiveness or Coupling Factor; dimensionless
$\overline{W}_S$ V_{Ws}	Weapon yield (KT) at the surface
$\overline{D}$ V_D	Depth of Facility (meters)
$\overline{L}$ V_L	Lateral Range (meters)
α	Exponent for Weapon Yield } dimensionless, constant
β	Exponent for Weapon Range } dimensionless, constant

ROCK DATA

(SIGUR)	$\bar{\sigma}_{UR}$	$V_{\sigma UR}$	Rock unconfined compressive stress (psi)
(KSR)	$\bar{K}_{sr}$	V_{Ksr}	Friction-dependent constant (dimensionless)
(ESUBR)	$\bar{E}_r$	V_{Er}	Equivalent Elastic Modulus (psi)
(NUR)	$\bar{\nu}_r$	$V_{\nu r}$	Poisson's Ratio (dimensionless)
(RCO)	$\bar{r}_{co}$	V_{rco}	Radius of Rock/Concrete Interface (inch)

CONCRETE & STEEL DATA

(FPC)	$\bar{f}'c$	$V_{f'c}$	Concrete unconfined compressive stress (psi)
(KSC)	$\bar{K}_{sc}$	V_{Ksc}	Friction-dependent constant (dimensionless) (concrete)
(ESUBC)	$\bar{E}_c$	V_{Ec}	Equivalent Elastic Modulus (psi) (concrete)
(NUC)	$\bar{\nu}_c$	$V_{\nu c}$	Poisson's Ratio (dimensionless) (concrete)
(RA)	$\bar{r}_a$	V_{ra}	Radius of Steel/Concrete Interface (inch)
(FY)	$\bar{f}_y$	V_{fy}	Yield Stress of Steel Liner (psi)
(HS)	$\bar{h}_s$	V_{hs}	Thickness of Steel Liner (inch)

CONCRETE STRENGTH PARAMETERS (Lognormal)

(MSC)	m_{sc}	V_{sc}	Failure strain in concrete at steel/concrete interface (dimensionless)
(UC)	U_c		Coefficient of Uncertainty for Concrete Strain

ROCK STRENGTH PARAMETERS (Lognormal)

(MSR)	m_{sr}	V_{sr}	Failure strain in rock at rock/concrete interface (dimensionless)
(UR)	U_r		Coefficient of Uncertainty for Rock Strain

```
      PROGRAM START(INPUT,OUTPUT)
C
C  THIS PROGRAM CALCULATES THE FAILURE PROBABILITY AND INITIAL COST
C  OF A DEEP-BURIED TUNNEL HAVING A COMPOSITE STEEL/CONCRETE LINER.
C  OPTION 0 ASSUMES FAILURE CAN BE RELATED TO CIRCUMFERENTIAL ROCK STRAIN.
C  OPTION 1 ASSUMES FAILURE CAN BE RELATED TO CIRCUMFERENTIAL STRAIN
C  IN THE STEEL/CONCRETE LINER.  OPTION 2 ASSUMES FAILURE CAN BE
C  RELATED TO EITHER ROCK STRAIN OR LINER STRAIN INDEPENDENTLY.  BOTH LOADS
C  AND STRENGTHS ARE ASSUMED LOGNORMAL. NEWMARK'S METHOD IS USED TO RELATE
C  MATERIAL  STRESSES AND STRAINS TO APPLIED FREE-FIELD PRESSURE.
C
      INTEGER OPTION,OUTPUT
    2 FORMAT (8G16.8)
      REAL A(100)
      REAL MUS,MUL
      REAL MN,ML,MS,MM
      REAL MUULC,  MUSC, MUULR,  INT7, MUSR, MLR, KSR, L, NC, NR, MLC
      REAL KW, NUR, KSC, NUC, MSR, MSC
      DATA OUTPUT/6LOUTPUT/
      READ*,OPTION,ND,DFL,DFH
      READ*,R,MM,BB,C,E,F,G,HH
      READ*,AO,VA,KW,VKW,WS,VWS,D,VD,L,VL,ALPHA,BETA
      READ*,SIGUR,VSIGUR,KSR,VKSR,ESUBR,VSUBER,NUR,VNUR,RCO,VRCO
      READ*,FPC,VFPC,KSC,VKSC,ESUBC,VSUBEC,NUC,VNUC,RA,VRA,FY,VFY,HS,VHS
      WRITE(OUTPUT,2001)
 2001 FORMAT(1H1)
      PRINT*,'INPUT DATA'
      PRINT*,'   '
      IF(OPTION-1)3,4,5
    3 PRINT*,' OPTION 0   * * *  ROCK STRAIN IS SINGLE FAILURE MODE'
      PRINT*,'   '
      GO TO 6
    4 PRINT*,' OPTION 1   * * *  CONCRETE STRAIN IS SINGLE FAILURE MODE'
      PRINT*,'   '
      GO TO 6
    5 PRINT*,' OPTION 2   * * *  ROCK STRAIN AND CONCRETE STRAIN'
      PRINT*,'                    ARE POSSIBLE FAILURE MODES'
      PRINT*,'   '
    6 CONTINUE
      PRINT*,'   '
      PRINT*,'NO. OF DEPTHS ',ND,'  RATIO MIN DEPTH : MEAN DEPTH ',DFL,'
     1  RATIO MAX DEPTH : MEAN DEPTH ',DFH
      PRINT*,'   '
      PRINT*,'COST DATA COEFFICIENTS'
      PRINT*,' TUNNEL LENGTH      VERTICAL EXCAVATION COSTS       VENTING
     1COSTS    HORIZONTAL EXCAVATION COSTS      STEEL LINER    CONCRETE LI
     2NER'
      PRINT*,'   (R,METERS)          (MM)              (BB)              (C)
     1            (E)                (F)               (G)             (HH)'
      PRINT 2,R,MM,BB,C,E,F,G,HH
      PRINT*,'   '
      PRINT*,'APPLIED LOAD DATA- MEAN, COEFFICIENT OF VARIATION'
      PRINT*,'        ROCK TRANSMISSIBILITY         WEAPON EFFECTIVENESS FAC
```

```
      1TOR     WEAPON YIELD AT SURFACE (KT)          MEAN DEPTH (METERS)  '
       PRINT*,'       (AO)             (VA)               (KW)              (VKW
      1)          (WS)             (VWS)              (D)              (VD)'
       PRINT 2,AO,VA,KW,VKW,WS,VWS,D,VD
       PRINT*,'        LATERAL RANGE (METERS)        WEAPON YIELD     WEAPON R
      1ANGE '
       PRINT*,'        (L)              (VL)          EXP (ALPHA)       EXP (BE
      1TA) '
       PRINT 2,L,VL,ALPHA,BETA
       PRINT*,'  '
       PRINT*,'ROCK DATA: MEAN; COEFFICIENT OF VARIATION'
       PRINT*,'       UNCONF COMP STRESS (PSI)           FRICTION DEP. CONSTAN
      1T         ELASTIC MODULUS (PSI)'
       PRINT*,'       (SIGUR)           (VSIGUR)            (KSR)            (VKSR
      1)         (ESUBR)           (VSUBER)'
       PRINT 2,SIGUR,VSIGUR,KSR,VKSR,ESUBR,VSUBER
       PRINT*,'           POISSONS RATIO              OUTSIDE RADIUS OF CONC
      1(IN)'
       PRINT*,'       (NUR)             (VNUR)            (RCO)             (VRCO
      1)'
       PRINT 2,NUR,VNUR,RCO,VRCO
       PRINT*,'  '
       PRINT*,'CONCRETE DATA- MEAN, COEFFICIENT OF VARIATION'
       PRINT*,'       UNCONF COMP STRESS (PSI)           FRICTION DEP. CONSTAN
      1T         ELASTIC MODULUS (PSI)                 POISSONS RATIO'
       PRINT*,'       (FPC)             (VFPC)            (KSC)             (VKSC
      1)         (ESUBC)           (VSUBFC)          (NUC)             (VNUC)'
       PRINT 2,FPC,VFPC,KSC,VKSC,ESUBC,VSUBEC,NUC,VNUC
       PRINT*,'  '
       PRINT*,'STEEL DATA- MEAN, COEFFICIENT OF VARIATION'
       PRINT*,'    OUTER RADIUS OF STEEL (IN)               YIELD STRESS (PSI)
      1          LINER THICKNESS (IN)'
       PRINT*,'       (RA)              (VRA)             (FY)              (VFY)
      1         (HS)              (VHS)'
       PRINT 2,RA,VRA,FY,VFY,HS,VHS
       IF(OPTION-1)10,15,20
    10 READ*,MSR,VSR,UR
       PRINT*,'  '
       PRINT*,'OPTION 0   ROCK STRENGTH PROPERTIES'
       PRINT*,'  MEAN STRAIN         COEFF VAR          COEFF UNC'
       PRINT*,'      (MSR)             (VSR)              (UR)'
       PRINT 2,MSR,VSR,UR
       U = UR
       GO TO 30
    15 READ*,MSC,VSC,UC
       PRINT*,'  '
       PRINT*,'OPTION 1   CONCRETE STRENGTH PROPERTIES'
       PRINT*,'  MEAN STRAIN         COEFF VAR          COEFF UNC'
       PRINT*,'      (MSC)             (VSC)              (UC)'
       PRINT 2,MSC,VSC,UC
       U = UC
       GO TO 30
    20 READ*,MSC,VSC,UC,MSR,VSR,UR
```

```
      PRINT*,'   '
      PRINT*,'OPTION 2   CONCRETE STRENGTH PROPERTIES'
      PRINT*,'  MEAN STRAIN         COEFF VAR        COEFF UNC'
      PRINT*,'     (MSC)             (VSC)             (UC)'
      PRINT 2,MSC,VSC,UC
      PRINT*,'   '
      PRINT*,'OPTION 2   ROCK STRENGTH PROPERTIES'
      PRINT*,'  MEAN STRAIN         COEFF VAR        COEFF UNC'
      PRINT*,'     (MSR)             (VSR)             (UR)'
      PRINT 2,MSR,VSR,UR
      PRINT*,'   '
   30 CONTINUE
      DL = DFL*D
      DH = DFH*D
      DELD = 0.0
      IF(ND.EQ.1) GO TO 31
      XND = FLOAT(ND - 1)
      DELD = (DL - DH)/XND
   31 CONTINUE
      DO 200 JJ = 1,ND
      WRITE(OUTPUT,2001)
      XJ = FLOAT( JJ - 1)
      D = DH + DELD*XJ
C
C  CALCULATE PARAMETERS OF COMBINATION VARIABLES FROM BASIC INPUT DATA.
C
      NC=(1.-2.*NUC)/(1.-NUC)
      EC=ESUBC/(1-NUC**2)
      RCA=RCO/RA
      SIGRA=HS*FY/RA
      SIGRC=((FPC+(KSC-1.)*SIGRA)*RCA**(KSC-1.)-FPC)/(KSC-1.)
      NR=(1.-2.*NUR)/(1.-NUR)
      ER=ESUBR/(1.-NUR**2)
      VXSQ = 4.*((D*D/(D*D+L*L)*VD)**2 + (L*L/(D*D+L*L)*VL)**2)
      VWUSQ=VKW**2+VWS**2
      P0=A0*(KW*WS)**ALPHA*(D*D+L*L)**BETA*(1.+VWUSQ)**(.5*ALPHA*(ALPHA-
     11.))*(1.+VXSQ)**(.5*BETA*(BETA-1.))
      MLR=(NR*SIGRC+(SIGUR+(KSR-1.)*SIGRC)**(-2./(KSR-1.))*(SIGUR+(2.*P0
     1-SIGUR)*(KSR-1.)/(KSR+1.))**((KSR+1.)/(KSR-1.)))/ER
      MLC=RCA**2*MLR+(SIGRA-RCA**2*SIGRC)*NC/EC
      VSIGRA=SQRT(VHS**2+VFY**2+VRA**2)
      VNC=NUC*VNUC/NC/(1.-NUC)**2
      VRCA=SQRT(VRCO**2+VRA**2)
      VLSQ = (1.+VA**2)*(1.+VWUSQ)**ALPHA**2*(1.+VXSQ)**BETA**2-1.
      VP0 = SQRT(VLSQ)
      RCRCA=RCA**(KSC-2.)*(FPC+(KSC-1.)*SIGRA)
      RCSGRA=RCA**(KSC-1.)
      RCKSC=RCA**(KSC-1.)*ALOG(RCA)*(SIGRA*(KSC-1.)+FPC)/(KSC-1.)-FPC*RC
     1A**(KSC-1.)/(KSC-1.)**2+FPC/(KSC-1.)**2
      RCFPC=(RCA**(KSC-1.)-1.)/(KSC-1.)
      SSRCSQ=(RCFPC*FPC*VFPC)**2+(RCKSC*KSC*VKSC)**2+(RCSGRA*SIGRA*VSIGR
     1A)**2+(RCRCA*RCA*VRCA)**2
      VSIGRC=SQRT(SSRCSQ)/SIGRC
```

```
      VNR=NUR*VNUR/NR/(1.-NUR)**2
      VEC=SQRT(VSUBEC**2+4.*NUC**4*(VNUC/(1.-NUC**2))**2)
      VER=SQRT(VSUBER**2+4.*NUR**4*(VNUR/(1.-NUR**2))**2)
      PRINT*,'PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VA
     1RIATION'
      PRINT*,'        PO             VPO              SIGRC          VSIGRC
     1        ER             VER              RCA            VRCA'
      PRINT 2,PO,VPO,SIGRC,VSIGRC,ER,VER,RCA,VRCA
      PRINT*,'        NR             VNR              NC             VNC
     1        EC             VEC              SIGRA          VSIGRA'
      PRINT 2,NR,VNR,NC,VNC,EC,VEC,SIGRA,VSIGRA
      PRINT*,'  '
      IF(OPTION-1)40,45,40
   40 CONTINUE
C
C       *****  OPTION 0  *****
C  ROCK STRAIN IS SINGLE FAILURE MODE.
C
C  DETERMINE THE VARIANCE OF THE ALLOWABLE LOAD CAUSING FAILURE.
C
      SRI = ER*MSR - NR*SIGRC
      YRI = SIGUR + (KSR-1.)*SIGRC
      MS = .5*(KSR+1.)/(KSR-1.)*(SRI**((KSR-1.)/(KSR+1.)))*(YRI**(2./
     1 (KSR+1.))) - SIGUR/(KSR-1.)
      Q = .5*(YRI/SRI)**(2./(KSR+1.))
      PER = Q*MSR
      PMSR = Q*ER
      PNR = -Q*SIGRC
      PSIGRC = -Q*NR
      PSIGUR = (1./(KSR-1.))*((SRI**((KSR-1.)/(KSR+1.)))*(2./(KSR+1.))
     1 *YRI**((1.-KSR)/(1.+KSR)) - 1.)
      PKSR = SIGUR/(KSR-1.)**2 + 2.*MS/(KSR+1.)*(-1./(KSR-1.)
     1 + ALOG(SRI)/(KSR+1.) + 1./YRI - ALOG(YRI)/(KSR+1.))
      SSSQ = (PER*ER*VER)**2 + (PMSR*MSR*VSR)**2 + (PNR*NR*VNR)**2
     1 + (PSIGRC*SIGRC*VSIGRC)**2 + (PSIGUR*SIGUR*VSIGUR)**2
     2 + (PKSR*KSR*VKSR)**2
      RMS = MS
      RSSQ = SSSQ
      VRMS = SQRT(RSSQ)/RMS
      PRINT*,'ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION'
      PRINT*,' MS = ',RMS,'   VMS = ',VRMS
      PRINT*,'  '
C
C  DETERMINE THE CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE LOAD VARIANCE.
C
      SSQI = 1./SSSQ
      FER = SSQI*(PER*ER*VER)**2
      FMSR = SSQI*(PMSR*MSR*VSR)**2
      FNR = SSQI*(PNR*NR*VNR)**2
      FSIGRC = SSQI*(PSIGRC*SIGRC*VSIGRC)**2
      FSIGUR = SSQI*(PSIGUR*SIGUR*VSIGUR)**2
      FKSR = SSQI*(PKSR*KSR*VKSR)**2
      PRINT*,'CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD V
```

```
     1ARIANCE'
      PRINT*,'      ER                 MSR                NR                SIGRC
     1            SIGUR                KSR'
      PRINT 2,FER,FMSR,FNR,FSIGRC,FSIGUR,FKSR
      PRINT*,'   '
      IF(OPTION-1.LT.0) GO TO 47
   45 CONTINUE
C
C          *****  OPTION 1  *****
C     CONCRETE STRAIN IS SINGLE FAILURE MODE.
C
C     DETERMINE THE VARIANCE OF THE ALLOWABLE LOAD CAUSING FAILURE.
C
      SRI = ER/EC*((EC*MSC-NC*SIGRA)/RCA**2 + NC*SIGRC) - NR*SIGRC
      YRI = SIGUR + (KSR -1.)*SIGRC
      MS = .5*(KSR +1.)/(KSR -1.)*(SRI**((KSR -1.)/(KSR +1.))*
     1YRI**(2./(KSR +1.))) - SIGUR/(KSR -1.)
      Q = .5*(YRI/SRI)**(2./(KSR +1.))
      PKSR = SRI**((KSR -1.)/(KSR +1.))/(KSR -1.)*(YRI**(2./(KSR +1.))
     1*(ALOG(SRI/YRI)/(KSR +1.) -1./(KSR -1.)) +YRI**((1.- KSR)/(1.
     1+ KSR))*SIGRC)+ SIGUR/(KSR -1.)**2
      PSIGUR = ((SRI/YRI)**((KSR -1.)/(KSR +1.)) -1.)/(KSR -1.)
      PER = Q/EC*((EC*MSC-NC*SIGRA)/RCA**2 + NC*SIGRC)
      PEC = Q*ER*NC/EC**2*(SIGRA/RCA**2 - SIGRC)
      PMSC = Q*ER/RCA**2
      PNC = Q*ER/EC*(SIGRC - SIGRA/RCA**2)
      PSIGRA = -Q*ER*NC/EC/RCA**2
      PRCA = -2.*Q*ER/EC*(EC*MSC - NC*SIGRA)/RCA**3
      PNR = -Q*SIGRC
      PSIGRC = Q*(ER*NC/EC - NR) +(SRI/YRI)**((KSR -1.)/(KSR +1.))
      SSSQ = (PKSR*KSR*VKSR)**2 + (PSIGUR*SIGUR*VSIGUR)**2 + (PER*ER
     1*VER)**2 + (PEC*EC*VEC)**2 + (PMSC*MSC*VSC)**2 + (PNC*NC*VNC)**2
     1+(PSIGRA*SIGRA*VSIGRA)**2 + (PRCA*RCA*VRCA)**2 + (PNR*NR*VNR)**2
     1+ (PSIGRC*SIGRC*VSIGRC)**2
      CMS = MS
      CSSQ = SSSQ
      VCMS = SQRT(CSSQ)/CMS
      PRINT*,'ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION'
      PRINT*,' MS = ',CMS,'   VMS = ',VCMS
      PRINT*,'   '
C
C     DETERMINE THE CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE LOAD VARIANCE.
C
      SSQI = 1.0/SSSQ
      FKSR  =SSQI*(PKSR*KSR*VKSR)**2
      FSIGUR=SSQI*(PSIGUR*SIGUR*VSISUR)**2
      FER   =SSQI*(PER*ER*VER)**2
      FEC   =SSQI*(PEC*EC*VEC)**2
      FMSC  =SSQI*(PMSC*MSC*VSC)**2
      FNC   =SSQI*(PNC*NC*VNC)**2
      FSIGRA=SSQI*(PSIGRA*SIGRA*VSIGRA)**2
      FRCA  =SSQI*(PRCA*RCA*VRCA)**2
      FNR   =SSQI*(PNR*NR*VNR)**2
```

```
      FSIGRC=SSQI*(PSIGRC*SIGRC*VSIGRC)**2
      PRINT*,'CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LO
     1AD VARIANCE'
      PRINT*,'      KSR             SIGUR           ER              EC
     1      MSC'
      PRINT 2,FKSR,FSIGUR,FER,FEC,FMSC
      PRINT*,'      NC              SIGRA           RCA             NR
     1      SIGRC'
      PRINT 2,FNC,FSIGRA,PRCA,FNR,FSIGRC
      PRINT*,'   '
   47 CONTINUE
      IF(OPTION-1.GT.0) GO TO 50
C
C  EVALUATE THE COMPONENT FAILURE PROBABILITY FOR OPTIONS 0 OR 1.
C
      VSSQ = SSSQ/MS**2
      MUS = ALOG(MS/SQRT(1. + VSSQ))
      ML = P0
      MUL = ALOG(ML/SQRT(1. + VLSQ))
      PF = PROB((MUL + ALOG(U)  - MUS)/SQRT(ALOG((1.+VLSQ)*(1.+VSSQ))))
      GO TO 60
   50 CONTINUE
C
C     *****  OPTION 2  *****
C  ROCK STRAIN AND CONCRETE STRAIN ARE POSSIBLE FAILURE MODES.
C
C  CALCULATE THE LOGNORMAL LOAD AND STRENGTH PARAMETERS.
C
      VSRSQ = RSSQ/(RMS**2)
      SIGSR = SQRT(ALOG(1.+VSRSQ))
      MUSR = ALOG(RMS/SQRT(1.+VSRSQ))
      VSCSQ = CSSQ/(CMS**2)
      SIGSC = SQRT(ALOG(1.+VSCSQ))
      MUSC = ALOG(CMS/SQRT(1.+VSCSQ))
      MUULR = ALOG(UR*P0/SQRT(1.+VLSQ))
      SIGLR = SQRT(ALOG(1.+VLSQ))
      MUULC = ALOG(UC*P0/SQRT(1.+VLSQ))
      SIGLC = SIGLR
C
C  EVALUATE THE SYSTEM FAILURE PROBABILITY BY NUMERICAL INTEGRATION.
C
      NN=15
      TL = -9.5
      TU = 9.5
      M=6*NN+1
      XM=FLOAT(6*NN)
      H=(TU-TL)/XM
      DO 100 I=1,M
      XI=FLOAT(I)
      T=TL+H*(XI-1.)
      TC=(SIGLC*T+MUULC -MUSC)/SIGSC
      TR=(SIGLR*T+MUULR - MUSR)/SIGSR
      IF(TC.LT.-10.) GO TO 51
```

```
      IF(TC.GT.10.) GO TO 52
      PROBTC = PROB(TC)
      GO TO 53
   51 PROBTC = 0.0
      GO TO 53
   52 PROBTC = 1.0
   53 CONTINUE
      IF(TR.LT.-10.) GO TO 54
      IF(TR.GT.10.) GO TO 55
      PROBTR = PROB(TR)
      GO TO 56
   54 PROBTR = 0.0
      GO TO 56
   55 PROBTR = 1.0
   56 CONTINUE
      A(I)=(PROBTC + PROBTR - PROBTC*PROBTR) * EXP(-.5*T**2)
  100 CONTINUE
      PF  = .3989422804014 33*INTT(NN,H,A)
   60 CONTINUE
C
C  CALCULATE PROBABILITY THAT ELASTIC/PLASTIC INTERFACE IS IN THE ROCK
C  AS REQUIRED BY NEWMARK*S EQUATION.
C
      MN = 2.*PO - SIGUR - (KSR + 1.)*SIGRC
      SN = SQRT(4.*(PO*VPO)**2 + (SIGUR*VSIGUR)**2 + (SIGRC*KSR*VKSR)
     1**2 +((KSR +1.)*SIGRC*VSIGRC)**2)
      PR = PROB(MN/SN)
C
C  CALCULATE INITIAL COST AS A FUNCTION OF DEPTH.
C
      CI = D*(MM +BB*D + C) + R*(E*(RA*RCA)**2*D**F + C + G*RA*HS
     1+HH*RA**2*(RCA**2 -1.))
      PRINT*,'   '
      PRINT*,' DEPTH(METERS)    FAILURE PROB.  INIT COST ($)   PROB THAT
     1 PLASTIC I/F IS IN ROCK'
      PRINT 2,D,PF,CI,PR
      PRINT*,'   '
  200 CONTINUE
      STOP
      END
```

```
F..CTION PROB        74/74    OPT=1 ROUND=+-*/              . TN 4.6  420         77/01/24. 16.53.21         PAGE    15

      FUNCTION PROB(X)
      IF(X.LE.5.0.AND.X.GE.-5.0)GO TO 20
      Y = 1.0
      SUM = 1.0
      PN = X**2/3. + 2.0
      NN = IFIX(PN)
      DO 5 J=1,NN
      XJ = FLOAT(J)
      Y = -Y*(2.0*XJ - 1.0)/X**2
      SUM = SUM + Y
    5 CONTINUE
      PRB = .398942280401433*SUM*EXP(-X**2/2.)/X
      IF(X.LT.-5.0) GO TO 10
      PP = 1.0 - PRB
      GO TO 40
   10 PP = -PRB
      GO TO 40
   20 Y=1.
      SUM=1.
      RN = 12. + 2.*X**2
      N=IFIX(RN)
      DO 30 I=2,N
      XI=FLOAT(I)
      Y=-.5*Y*X**2/(XI-1.)
   30 SUM=SUM+Y/(2.*XI-1.)
      PP = .5 + .398942280401433*X*SUM
   40 PROB=PP
      RETURN
      END
```

```
FUNCTION INT7        74/74    OPT=1 ROUND=+-*/              FTN 4.6  420          77/01/24. 16.53.21        PAGE     1

              REAL FUNCTION INT7(NN,H,A)
              REAL A(100)
              SUM1 = 0.0
              SUM2 = 0.0
              DO 10 I=1,NN
              SUM2=SUM2+A(6*I-5)+A(6*I-3)+A(6*I-1)+A(6*I+1)
           10 SUM1=SUM1+5.0*(A(6*I-4)+A(6*I))+6.0*(A(6*I-2))
              INT7= 0.3*H*(SUM1+SUM2)
              RETURN
              END
```

```
INPUT DATA

OPTION 2  * * *  ROCK STRAIN AND CONCRETE STRAIN
                 ARE POSSIBLE FAILURE MODES

NO. OF DEPTHS 9  RATIO MIN DEPTH : MEAN DEPTH .8  RATIO MAX DEPTH : MEAN DEPTH 2.4

COST DATA COEFFICIENTS
 TUNNEL LENGTH     VERTICAL EXCAVATION COSTS        VENTING COSTS   HORIZONTAL EXCAVATION COSTS       STEEL LINER      CONCRETE LINER
  (R,METERS)         (MM)             (BB)              (C)              (E)              (F)             (G)              (HH)
 30.500000        305.00000        2.0000000        99.000000        .20200000        .80000000E-01   2.3300000        .17800000E-01

APPLIED LOAD DATA- MEAN, COEFFICIENT OF VARIATION
     ROCK TRANSMISSIBILITY         WEAPON EFFECTIVENESS FACTOR       WEAPON YIELD AT SURFACE (KT)         MEAN DEPTH (METERS)
         (AO)            (VA)             (KW)             (VKW)            (WS)             (VWS)            (D)              (VD)
 41100000.        .67400000        .40000000E-01    0.               25000.000        0.               500.00000        0.
     LATERAL RANGE (METERS)        WEAPON YIELD     WEAPON RANGE
         (L)             (VL)      EXP (ALPHA)      EXP (BETA)
 0.               0.               .57330000        -.86000000

ROCK DATA: MEAN; COEFFICIENT OF VARIATION
    UNCONF COMP STRESS (PSI)          FRICTION DEP. CONSTANT            ELASTIC MODULUS (PSI)
      (SIGUR)          (VSIGUR)           (KSR)            (VKSR)           (ESUBR)          (VSUBER)
 24600.000        .22600000        2.4500000        .20500000        9310000.0        .19400000
          POISSONS RATIO              OUTSIDE RADIUS OF CONC (IN)
       (NUR)           (VNUR)            (RCO)            (VRCO)
 .31000000        0.               96.000000        .20000000E-01

CONCRETE DATA- MEAN, COEFFICIENT OF VARIATION
    UNCONF COMP STRESS (PSI)          FRICTION DEP. CONSTANT            ELASTIC MODULUS (PSI)             POISSONS RATIO
       (FPC)           (VFPC)            (KSC)            (VKSC)           (ESUBC)          (VSUBEC)         (NUC)            (VNUC)
 7020.0000        .55600000E-01    4.1000000        .34100000        5000000.0        .40000000E-01    .30000000        0.

STEEL DATA- MEAN, COEFFICIENT OF VARIATION
   OUTER RADIUS OF STEEL (IN)            YIELD STRESS (PSI)              LINER THICKNESS (IN)
       (RA)            (VRA)             (FY)             (VFY)            (HS)             (VHS)
 84.000000        .50000000E-02    52800.000        .60000000E-01    1.2500000        .30000000E-02

OPTION 2   CONCRETE STRENGTH PROPERTIES
  MEAN STRAIN      COEFF VAR        COEFF UNC
     (MSC)            (VSC)            (UC)
 .40000000E-01    .59000000        1.0000000

OPTION 2   ROCK STRENGTH PROPERTIES
  MEAN STRAIN      COEFF VAR        COEFF UNC
     (MSR)            (VSR)            (UR)
 .50000000E-01    .55000000        1.0000000
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PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
     PO            VPO           SIGRC         VSIGRC         ER            VER           RCA           VRCA
  10903,102      ,67400000     2349,8036     ,19517707     10299812,     ,19400000     1,1428571     ,20615528E-01
     NR            VNR            NC            VNC           EC            VEC          SIGRA         VSIGRA
  ,55072464      0,            ,57142857     0,            5494505,5     ,40000000E-01  785,71429    ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397   VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
     ER            MSR            NR           SIGRC          SIGUR          KSR
 ,96063482E-01   ,77211190     0,            ,61396836E-06  ,15494109E-01  ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699   VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
     KSR          SIGUR           ER             EC            MSC
  ,10247453     ,65670434E-01  ,81440115E-01  ,12059535E-06  ,74438476
     NC           SIGRA           RCA            NR           SIGRC
  0,            ,32430554E-07  ,36204800E-02  0,            ,24095233E-02

 DEPTH(METERS)    FAILURE PROB,   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  1200,0000       ,18217149E-02   3476575,7       ,24563518


PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
     PO            VPO           SIGRC         VSIGRC         ER            VER           RCA           VRCA
  12663,286      ,67400000     2349,8036     ,19517707     10299812,     ,19400000     1,1428571     ,20615528E-01
     NR            VNR            NC            VNC           EC            VEC          SIGRA         VSIGRA
  ,55072464      0,            ,57142857     0,            5494505,5     ,40000000E-01  785,71429    ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397   VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
     ER            MSR            NR           SIGRC          SIGUR          KSR
 ,96063482E-01   ,77211190     0,            ,61396836E-06  ,15494109E-01  ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699   VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
     KSR          SIGUR           ER             EC            MSC
  ,10247453     ,65670434E-01  ,81440115E-01  ,12059535E-06  ,74438476
     NC           SIGRA           RCA            NR           SIGRC
  0,            ,32430554E-07  ,36204800E-02  0,            ,24095233E-02

 DEPTH(METERS)    FAILURE PROB,   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  1100,0000       ,35737657E-02   2975481,2       ,34140464
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PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
      PO             VPO            SIGRC          VSIGRC            ER              VER             RCA             VRCA
  14919,072       ,67400000       2349,8036       ,19517707       10299812,        ,19400000       1,1428571       ,20615528E-01
      NR             VNR              NC             VNC              EC              VEC            SIGRA           VSIGRA
  ,55072464       0,              ,57142857       0,              5494505,5        ,40000000E-01   785,71429       ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397    VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
      ER             MSR              NR            SIGRC           SIGUR            KSR
  ,96063482E-01   ,77211190       0,              ,61396836E-06   ,15494109E-01    ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699    VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
      KSR            SIGUR            ER              EC              MSC
  ,10247453       ,65670434E-01   ,81440115E-01   ,12059535E-06   ,74438476
      NC             SIGRA            RCA             NR             SIGRC
  0,              ,32430554E-07   ,36204800E-02   0,              ,24095233E-02

  DEPTH(METERS)     FAILURE PROB,   INIT COST ($)    PROB THAT PLASTIC I/F IS IN ROCK
  1000,0000         ,71197297E-02   2514325,9        ,44556485


PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
      PO             VPO            SIGRC          VSIGRC            ER              VER             RCA             VRCA
  17883,178       ,67400000       2349,8036       ,19517707       10299812,        ,19400000       1,1428571       ,20615528E-01
      NR             VNR              NC             VNC              EC              VEC            SIGRA           VSIGRA
  ,55072464       0,              ,57142857       0,              5494505,5        ,40000000E-01   785,71429       ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397    VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
      ER             MSR              NR            SIGRC           SIGUR            KSR
  ,96063482E-01   ,77211190       0,              ,61396836E-06   ,15494109E-01    ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699    VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
      KSR            SIGUR            ER              EC              MSC
  ,10247453       ,65670434E-01   ,81440115E-01   ,12059535E-06   ,74438476
      NC             SIGRA            RCA             NR             SIGRC
  0,              ,32430554E-07   ,36204800E-02   0,              ,24095233E-02

  DEPTH(METERS)     FAILURE PROB,   INIT COST ($)    PROB THAT PLASTIC I/F IS IN ROCK
  900,00000         ,14377916E-01   2093097,7        ,54905697
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PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
     PO            VPO           SIGRC         VSIGRC          ER             VER            RCA           VRCA
  21899,139     ,67400000      2349,8036     ,19517707      10299812,      ,19400000      1,1428571     ,20615528E-01
     NR            VNR            NC            VNC            EC             VEC            SIGRA         VSIGRA
  ,55072464     0,             ,57142857     0,             5494505,5      ,40000000E-01  785,71429     ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397   VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
     ER            MSR            NR            SIGRC          SIGUR          KSR
  ,96063482E-01  ,77211190     0,             ,61396836E-06  ,15494109E-01  ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699   VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
     KSR           SIGUR          ER            EC             MSC
  ,10247453     ,65670434E-01  ,81440115E-01  ,12059535E-06  ,74438476
     NC            SIGRA          RCA           NR             SIGRC
  0,            ,32430554E-07  ,36204800E-02  0,             ,24095233E-02

DEPTH(METERS)   FAILURE PROB,   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  800,00000     ,29329646E-01   1711780,1       ,64372760


PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
     PO            VPO           SIGRC         VSIGRC          ER             VER            RCA           VRCA
  27553,273     ,67400000      2349,8036     ,19517707      10299812,      ,19400000      1,1428571     ,20615528E-01
     NR            VNR            NC            VNC            EC             VEC            SIGRA         VSIGRA
  ,55072464     0,             ,57142857     0,             5494505,5      ,40000000E-01  785,71429     ,60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196,49061397   VMS = ,3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
     ER            MSR            NR            SIGRC          SIGUR          KSR
  ,96063482E-01  ,77211190     0,             ,61396836E-06  ,15494109E-01  ,11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295,28868699   VMS = ,3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
     KSR           SIGUR          ER            EC             MSC
  ,10247453     ,65670434E-01  ,81440115E-01  ,12059535E-06  ,74438476
     NC            SIGRA          RCA           NR             SIGRC
  0,            ,32430554E-07  ,36204800E-02  0,             ,24095233E-02

DEPTH(METERS)   FAILURE PROB,   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  700,00000     ,60052990E-01   1370350,2       ,72428525
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PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
      PO             VPO            SIGRC          VSIGRC           ER               VER              RCA           VRCA
  35918.786       .67400000       2349.8036      .19517707      10299812.        .19400000        1.1428571     .20615528E-01
      NR             VNR             NC             VNC             EC               VEC              SIGRA         VSIGRA
  .55072464       0.              .57142857      0.             5494505.5        .40000000E-01    785.71429     .60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196.49061397   VMS = .3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
      ER             MSR             NR             SIGRC           SIGUR            KSR
  .96063482E-01   .77211190       0.             .61396836E-06  .15494109E-01    .11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295.28868699   VMS = .3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
      KSR            SIGUR           ER             EC              MSC
  .10247453       .65670434E-01   .81440115E-01  .12059535E-06  .74438476
      NC             SIGRA           RCA            NR              SIGRC
  0.              .32430554E-07   .36204800E-02  0.             .24095233E-02

 DEPTH(METERS)     FAILURE PROB.   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  600.00000        .12200466       1068774.9       .78879407


PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION
      PO             VPO            SIGRC          VSIGRC           ER               VER              RCA           VRCA
  49148.856       .67400000       2349.8036      .19517707      10299812.        .19400000        1.1428571     .20615528E-01
      NR             VNR             NC             VNC             EC               VEC              SIGRA         VSIGRA
  .55072464       0.              .57142857      0.             5494505.5        .40000000E-01    785.71429     .60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
 MS = 96196.49061397   VMS = .3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE
      ER             MSR             NR             SIGRC           SIGUR            KSR
  .96063482E-01   .77211190       0.             .61396836E-06  .15494109E-01    .11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
 MS = 75295.28868699   VMS = .3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE
      KSR            SIGUR           ER             EC              MSC
  .10247453       .65670434E-01   .81440115E-01  .12059535E-06  .74438476
      NC             SIGRA           RCA            NR              SIGRC
  0.              .32430554E-07   .36204800E-02  0.             .24095233E-02

 DEPTH(METERS)     FAILURE PROB.   INIT COST ($)   PROB THAT PLASTIC I/F IS IN ROCK
  500.00000        .24094222       807003.33       .83796031
```

PARAMETERS OF COMBINED VARIABLES - MEAN; COEFFICIENT OF VARIATION

PO	VPO	SIGRC	VSIGRC	ER	VER	RCA	VRCA
72143.737	.67400000	2349.8036	.19517707	10299812.	.19400000	1.1428571	.20615528E-01
NR	VNR	NC	VNC	EC	VEC	SIGRA	VSIGRA
.55072464	0.	.57142857	0.	5494505.5	.40000000E-01	785.71429	.60282667E-01

ALLOWABLE LOAD IN ROCK - MEAN, COEFFICIENT OF VARIATION
MS = 96196.49061397   VMS = .3102455648579

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE ROCK LOAD VARIANCE

ER	MSR	NR	SIGRC	SIGUR	KSR
.96063482E-01	.77211190	0.	.61396836E-06	.15494109E-01	.11632990

ALLOWABLE LOAD IN CONC - MEAN, COEFFICIENT OF VARIATION
MS = 75295.28868699   VMS = .3515246131409

CONTRIBUTION OF EACH VARIABLE TO THE ALLOWABLE CONCRETE LOAD VARIANCE

KSR	SIGUR	ER	EC	MSC
.10247453	.65670434E-01	.81440115E-01	.12059535E-06	.74438476
NC	SIGRA	RCA	NR	SIGRC
0.	.32430554E-07	.36204800E-02	0.	.24095233E-02

DEPTH(METERS)	FAILURE PROB.	INIT COST ($)	PROB THAT PLASTIC I/F IS IN ROCK
400.00000	.44580243	584951.70	.87394850

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